HOUSATONIC RIVER FLOOD CONTROL

ANSONIA - DERBY
LOCAL PROTECTION

NAUGATUCK RIVER, CONNECTICUT

DESIGN MEMORANDUM NO.1

HYDROLOGY AND INTERIOR DRAINAGE

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS WALTHAM, MASS.

APRIL 1965

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SUBJECT: Ansonia-Derby Local Protection Project, Naugatuck River, Connecticut, Design Memorandum No. 1 - Hydrology and Interior Drainage

HQ, DA, CofEngrs, Washington, D. C., 20315, 16 June 1965

TO: Division Engineer, U. S. Army Engineer Division, New England

Approved as a basis for further planning.

FOR THE CHIEF OF ENGINEERS:

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DIVISION ENGINEER

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30 April 1965

SUBJECT: Ansonia-Derby Local Protection Project, Naugatuck River, Connecticut, Design Memorandum No. 1 - Hydrology and Interior Drainage

TO: Chief of Engineers
ATTN: ENGCW-E

There is submitted herewith for review and approval Design Memorandum No. 1, Hydrology and Interior Drainage for Ansonia-Derby Local Protection Project, Housatonic River Basin, in accordance with EM 1110-2-1150.

FOR THE DIVISION ENGINEER:

John Wm Leslie

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as (5 cys)

JOHN WM. LESLIE
Chief, Engineering Division

FLOOD CONTROL PROJECT

ANSONIA-DERBY LOCAL PROTECTION PROJECT
NAUGATUCK RIVER
HOUSATONIC RIVER BASIN
CONNECTICUT

DESIGN MEMORANDA INDEX

No.	Title	Submission Date	Approved
1	Hydrology and Interior Drainage	30 Apr 1965	
2	(Omitted)		
3	General Design (Including Real Estate)		
4	Concrete Materials		
5	Site Geology		
6	Embankment Foundations and Channel Improvements		
7	Floodwalls		
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ANSONIA-DERBY LOCAL PROTECTION PROJECT

NAUGATUCK RIVER
CONNECTICUT

DESIGN MEMORANDUM NO. 1

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ANSONIA-DERBY LOCAL PROTECTION PROJECT

NAUGATUCK RIVER
CONNECTICUT

DESIGN MEMORANDUM NO. 1

1. INTRODUCTION

a. Purpose. The purpose of this memorandum is to describe the hydrologic criteria applicable to the design and the interior drainage plan of the Ansonia-Derby Local Protection Project in Ansonia and Derby, Connecticut on the Naugatuck River and Beaver Brook. Included are sections on climatology, streamflow, floods of record, standard project floods and flood reductions afforded by other flood control projects. The Ansonia-Derby project is entirely separate from the adjoining Derby local protection project which has been recommended and is presently awaiting Congressional action.

b. Coordination with local authorities. The general design has been developed with the knowledge and concurrence of the officials of the cities of Ansonia and Derby who will be furnished copies of this design memorandum for review and comment.

PART I - HYDROLOGY

2. DESCRIPTION OF WATERSHED AND PROJECT

a. Naugatuck River watershed. The Naugatuck River, a principal tributary of the Housatonic River, is a rapidly flowing, nonnavigable stream. The watershed, located entirely within the western part of Connecticut, is about 50 miles long with a maximum width of 12 miles and a total drainage area of 312 square miles (see Basin Map, Plate No. 1-1). The headwaters of the Naugatuck River lie about 6 miles south of the Massachusetts line in the southwest corner of the town of Norfolk at an elevation of about 1,500 feet. The general direction of flow is southerly through the communities of Torrington, Thomaston, Waterbury, Naugatuck, Beacon Falls, Seymour, Ansonia and Derby. The Naugatuck River joins the Housatonic River in its tidal reach, about 12 miles from Long Island Sound. Between the headwaters and Torrington, the river falls approximately 900 feet in about 13 miles. The Naugatuck River below Torrington slopes at a rather uniform rate of about 14 feet per mile to tidewater in Ansonia.

b. Beaver Brook watershed. Beaver Brook, with a drainage area of 3.58 square miles, enters the Naugatuck River at river mile 1.55, about midway between Division Street and the NYNH&H railroad bridge in Ansonia. A portion of the drainage area is controlled by head-water reservoirs so that a net area of 2.89 square miles contributes to peak discharges at Central Street. The topography of the water-shed is hilly and therefore conducive to rapid runoff. Beaver Brook is 3 miles long and has a slope of 150 feet per mile. This relatively steep channel slope meets the Naugatuck River flood plain near Central Street, skirts the edge of the flood plain, following the toe of a ridge, and then for the last 0.2 mile of its course, cuts through the flood plain to the Naugatuck River.

c. Project description. The project is located within the cities of Ansonia and Derby, New Haven County, Connecticut on the Naugatuck River and Beaver Brook tributary. Maps of the Naugatuck River basin and the project site are shown on Plate Nos. 1-1 and 1-2. The flood protection project will extend 2 miles along the Naugatuck River, beginning 1,300 feet below the Division Street bridge at river mile 1.0 in the city of Derby, and terminating near the American Brass Company hydroelectric plant at river mile 3.0 in the city of Ansonia. A line of protection will also be constructed on the west side of Beaver Brook, from 400 feet above Central Street to its confluence with the Naugatuck River. The project will include some relocation and realignment of the river channel and construction of dikes, floodwalls, highway gate structures and interior drainage facilities, including pumping stations. The project will provide flood protection to properties on both sides of the Naugatuck River in the event of the standard project flood.

3. CLIMATOLOGY

a. General. The Naugatuck River basin has a variable climate characterized by frequent but usually short periods of precipitation. The basin lies in the path of the "prevailing westerlies" which often include cyclonic disturbances that cross the country from the west or southwest. It is also exposed to occasional coastal storms, some of tropical origin, that travel up the Atlantic seaboard. In late summer and autumn months these storms occasionally attain hurricane intensity. The southern portion of the basin, due to its proximity to the Atlantic coast, escapes the severity of cold and depth of snowfall experienced in the higher elevations in the northern part of the watershed.

b. Temperature. Average monthly temperatures in the Naugatuck River basin vary widely throughout the year. The mean annual temperature is approximately 47° F., ranging from about 50° F. near the coast to about 44° F. in the headwaters. The minimum temperature recorded in the basin was -25° F. at Waterbury in February 1943; the maximum recorded temperature was 105° F. which occurred at Waterbury in July 1926. Freezing temperatures can be expected from the middle of November until the end of March. The mean, maximum and minimum temperatures recorded each month at Bridgeport and Waterbury, Connecticut are reported in Table 1-1.

c. Precipitation. The mean annual precipitation over the Naugatuck River watershed is approximately 50 inches, varying from 45.86 inches at Bridgeport, near the mouth, to 53.48 inches at Norfolk in the northern end of the watershed. The maximum and minimum annual precipitations at Waterbury for 67 years through 1954 are 66.58 inches in 1901 and 31.21 inches in 1931. The rainfall gage at Waterbury was destroyed in August 1955 and was not replaced until 1957. However, annual precipitation for 1955 has been estimated at approximately 65 inches. At Bridgeport, representative of the southern portion of the watershed, the total precipitation for 1955 was 64.2 inches with 9.62 inches and 10.72 inches observed during August and October, respectively. Table 1-2 summarizes the precipitation records at Bridgeport and Waterbury.

d. Snowfall. The average annual snowfall over the watershed varies from about 35 inches near the coast to over 80 inches in the headwater region. Monthly and annual average snowfall for 35 years at Norfolk and 62 years at Bridgeport are tabulated in Table 1-3. Snow cover reaches a maximum depth in March with the water content often 4 to 6 inches.

e. Storms. The Housatonic River watershed has experienced storms of 4 general types, namely:

(1) Extratropical continental storms which move across the basin under the influence of the "prevailing westerlies".

(2) Extratropical maritime storms which originate and move northward along the eastern United States coast.

(3) Storms of tropical origin, some of which attain hurricane magnitude.

(4) Thunderstorms produced by local convective activity or by more general frontal activity.

TABLE 1-1

MONTHLY TEMPERATURE RECORD
(Degrees Fahrenheit)

	Bridgeport, Connecticut Elevation 7 feet, msl 66 Years of Record			Waterbury, Connecticut Elevation 340 feet, msl 67 Years of Record		
Month	Mean	Maximum	Minimum	Mean	Maximum	Minimum
January	29.5	68	-14	27.9	73	-19
February	29.9	70	-20	28.3	70	-25
March	37.8	85	1	37.1	87	0
April	48.1	97	9	48.3	92	11
May	57.0	95	26	59.4	96	26
June	67.5	99	34	68.0	101	33
July	73.1	103	44	72.9	105	41
August	71.3	101	38	70.8	104	35
September	65.2	98	32	64.2	103	25
October	54.5	90	20	53.5	94	17
November	43.5	80	8	42.3	84	2
December	32.3	67	-12	31.1	70	-12
ANNUAL	51.0	103	-20	50.3	105	-25

TABLE 1-2

MONTHLY PRECIPITATION RECORD
(In Inches)

	Bridgeport, Connecticut Elevation 7 feet, msl 66 Years of Record			Waterbury, Connecticut Elevation 340 feet, msl 67 Years of Record		
Month	Mean	Maximum	Minimum	Mean	Maximum	Minimum
January	3.76	7.88	0.51	3.87	10.06	0.84
February	3.47	6.32	0.85	3.52	10.00	0.43
March	4.18	9.64	0.29	4.08	9.46	0.17
April	3.90	9.41	0.69	3.72	11.51	0.66
May	3.78	10.18	0.49	3.95	8.08	0.13
June	3.29	8.48	0.06	3.59	11.25	0.54
July	4.05	18.77	0.45	4.34	18.10	1.36
August	4.45	13.29	0.20	4.30	9.48*	0.90
September	3.73	14.15	0.09	3.66	12.90	0.90
October	3.64	10.72	0.30	3.46	8.83*	0.20
November	3.78	7.60	0.81	3.81	8.74	0.78
December	3.83	9.85	0.33	3.90	9.82	0.82
ANNUAL	45.86	64.23	29.57	46.30	66.58	31.21

* Probably exceeded in 1955

TABLE 1-3

MEAN MONTHLY SNOWFALL
(Average Depth in Inches)

	Bridgeport, Connecticut Elevation 7 feet, msl 62 Years of Record Snowfall	Norfolk, Connecticut Elevation 1380 feet, msl 35 Years of Record Snowfall
January	8.7	18.5
February	10.1	20.9
March	6.9	15.6
April	1.3	5.7
May	0.0	0.3
June	0.0	0.0
July	0.0	0.0
August	0.0	0.0
September	0.0	0.0
October	0.0	0.3
November	1.5	6.2
December	6.5	11.9
ANNUAL	35.0	79.4

The most severe storms have been of tropical origin which occur during the late summer and early autumn. The five notable recent storms in the Naugatuck River basin occurred in March 1936, September 1938, December 1948, and August and October 1955. Of these, the storms of September 1938, and August and October 1955 were of tropical origin. The August 1955 storm, accompanying hurricane Diane, dumped about 13 inches of rain in the upper part of the watershed in about 48 hours. Mass curves of rainfall and isohyetal maps for storms of August and October 1955 are shown on Plate No. 1-4. A more detailed discussion of recent storms in the Naugatuck River basin is included in Appendix B, "Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River," June 1958.

4. RUNOFF

a. Discharge records. The U. S. Geological Survey (USGS) has published records of river stages and streamflows at 4 locations in the basin for various periods of time since 1918. The records are good to excellent, except during periods of ice when they are fair. Records have been lost during major floods, but were reproduced from records at dams, with peak discharges often determined from slope area measurements. Flow data at the gaging stations are summarized in Table 1-4. The mean daily discharges of the Naugatuck River at the USGS Station near Naugatuck from 1936-1955 are shown on Plate No. 1-13.

b. Streamflow data. The average annual runoff from the Naugatuck River basin, based on 33 years of record at the gaging station near Naugatuck, Connecticut is about 470 cfs, equivalent to 26 inches from the tributary drainage area of 246 square miles. This represents about 50 percent of the average annual precipitation, with one-third of the annual runoff normally occurring during the months of March and April. Mean, maximum and minimum monthly and annual runoff in cfs for the period of record at the Naugatuck gage is shown in Table 1-5.

5. HISTORY OF FLOODS

a. General. Floods in the Naugatuck River may result from intense rainfall over the watershed or from rainfall in conjunction with melting snow such as the flood of March 1936. Historic records indicate that floods may occur during any season of the year. The more critical floods develop from rainfall alone, when the intensity of the rainfall and antecedent conditions rather than the total volume determine the magnitude of the flood peaks. This was evident in the storm of August 1955 when the total rainfall was only about one-third greater than the

TABLE 1-4

STREAMFLOW RECORDS

Location	Drainage Area (sq.mi.)	Period of Record	Discharge (CFS) Mean*	Maximum**	Minimum
Naugatuck River near Thomaston	71.9	1930-1958	144	41,600	7
Leadmine Brook near Thomaston	24	1930-1958	48.0	10,400	0.08
Naugatuck River near Naugatuck***	246	1918-1924 1929-1955	470	106,000	24

* Through 1955
** Instantaneous discharge, August 1955
*** Destroyed in 1955 flood, gage installed near Beacon Falls in 1956

TABLE 1-5

MONTHLY RUNOFF
January 1919 - September 1955
(cubic feet per second)

Naugatuck River Near
Naugatuck, Connecticut
Drainage Area - 246 square miles

Month	Mean	Maximum	Minimum
January	563	1,449	132
February	506	1,393	204
March	1017	2,155	459
April	891	1,921	393
May	586	1,076	255
June	345	799	133
July	219	957	88
August	259	2,920	76
September	226	1,301	68
October	200	566	55
November	372	1,176	64
December	496	1,066	113
ANNUAL	473		

storm of December 1948, but the flood peaks were almost four times the magnitude. The quick development of floods is due to the many short, steep tributaries which empty into the main channel concurrently. This is illustrated by the fact that major floods have crested along the entire length of the Naugatuck River within a period of 5 to 8 hours.

b. Floods of record. The Naugatuck River basin has experienced 5 major floods within the past 30 years. These floods are briefly described in the following paragraphs and their respective discharges are shown in Table 1-6. Water surface profiles of the August and October 1955 floods are shown on Plate No. 1-2.

(1) March 1936. The March 1936 floods resulted from the storms of 9-13 and 16-22 March. The hydrographs at all gaging stations show two rises separated by a recession almost to base flow. Although the rainfall in each storm averaged about 3 inches over the Naugatuck River basin, the first rise was more than three times as great as the second in the lower basin. This was due to two basic factors: (a) most of the rainfall in the first storm fell within 24 hours while the second was over a 48-hour period; and (b) the first flood was augmented by snowmelt from the higher elevations and minor ice jams.

(2) September 1938. The storm producing the flood started with light rain, gradually increasing in intensity over a 4-day period and ending with a heavy downpour associated with the hurricane that struck New England on the 21st of September. The rainfall pattern was especially conducive to high peak discharges due to the filling of ponds, lakes and swamps and satisfying of initial soil moisture deficiencies before the intense rainfall occurred. Moreover, rainfall during the previous month was heavier than normal, thereby, reducing the initial storage and infiltration losses.

(3) December 1948. The storm which produced this flood deposited 4 to 9 inches of rain in approximately 48 hours over the Naugatuck River valley. This flood crested at stages slightly higher than the September 1938 flood and was the maximum of record prior to 1955. The runoff from rainfall was augmented by snowmelt and frozen ground conditions.

(4) August 1955. The greatest flood of record in the Naugatuck River basin occurred in August 1955. Peak discharges were 3 to 4 times the magnitude of any other flood. Between the 11th and 15th of August,

TABLE 1-6

MAJOR FLOODS
HOUSATONIC AND NAUGATUCK RIVERS

Date of Flood		Water Surface Elevation at Division Street Bridge (feet,msl)	Contribution to Maximum Discharge in Housatonic River (cfs)			Water Surface Elevation at Shelton, Conn. (feet,msl)	Concurrent Tide Condition at Derby, Conn.
			Naugatuck River	Housatonic River	Total		
Mar	1936	21±	27,000	60,000	87,000	18.0	-
Sept	1938	21.3	30,000	60,000	90,000	19.6	Abnormal*
Dec	1948	18.8	32,000	50,000	82,000	17.2	Near normal high
Aug	1955	27.9	112,000	40,000	152,000	21.0	Normal high
Oct	1955	23.8	40,000	75,000	115,000	21.0	Abnormal**

* Abnormal high tide due to hurricane occurred 21 September 1938 at 8 P.M. Maximum discharge occurred 22 September at 1 A.M. Maximum stage recorded concurrently with abnormal high tide.

** Stages of Housatonic River at Devon, Connecticut near the mouth remained 2 to 3 feet above predicted tides throughout the flood. Maximum river stage occurred during abnormally high predicted low tide.

hurricane Connie brought 4 to 8 inches of rainfall to the basin, but very little runoff occurred due to the unusually dry antecedent conditions. However, when hurricane Diane produced record amounts of precipitation, between 10 and 13 inches, on regions previously saturated by the rainfall of hurricane Connie runoff of record proportions occurred. Most of the rain fell in a 24-hour period between 7 a.m. 18 August and 7 a.m. 19 August 1955. The failure of many dams and bridges contributed substantially to peak discharges. Natural and modified profiles of the 1955 flood in Ansonia are shown on Plate No. 1-2.

(5) October 1955. The flood, occurring as the result of the storm of 14-17 October 1955, was confined to southwestern Connecticut and western Massachusetts. It was the second largest flood of record in lower portions of the Naugatuck River basin. Abnormally high tides contributed to the flood stages in the communities of Derby and Ansonia.

6. FLOOD FREQUENCY

The frequency or percent chance of occurrence of flood discharges was determined from records of all gaging stations in the Naugatuck and adjacent river basins. The frequency analyses were made in accordance with procedures set forth in ER 1110-2-1450. Based on a regional analysis, the skew coefficient adopted for the Naugatuck River basin was 1.0. Natural and modified peak discharge frequency curves at Maple Street bridge, applicable to the Ansonia reach, are shown on Plate No. 1-5.

7. ANALYSIS OF FLOODS

a. General. Analyses of all recent floods and the development of the standard project flood for the entire Naugatuck River basin were made and reported in the "Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River," dated 30 June 1958.

For purposes of establishing the value of upstream reservoirs and for hydrologic and hydraulic analyses, the Naugatuck River basin was divided into subdivisions and routing reaches. The component hydrographs for the basin subdivisions were either obtained from gaging station records or developed synthetically from the observed hydrographs and rainfall data of comparable gaged areas. Component hydrographs were routed downstream to determine their contributions to the flood peaks at Ansonia using the Progressive Average-Lag Method of flood routing.

b. Effect of tide and Housatonic River discharge. Water surface levels in the Ansonia-Derby area are affected by concurrent flows in the Housatonic River at the mouth of the Naugatuck River and the flow in the Naugatuck River. Stages at the confluence of the Naugatuck and Housatonic Rivers are also influenced by tide conditions in Long Island Sound which in turn affects stages in the Ansonia-Derby area. Table 1-6 shows a tabulation of the relationships between discharge, stage and coastal tides during recent major floods.

8. EFFECT OF RESERVOIRS

The flood control plan for the Naugatuck River includes 7 flood control dams and reservoirs, namely, Thomaston, Hall Meadow Brook, East Branch, Northfield Brook, Black Rock, Hancock Brook and Hop Brook. At present Thomaston, Hall Meadow Brook and East Branch Dams are completed. Northfield Brook and Hancock Brook Dams are under construction, and Black Rock and Hop Brook Dams are presently being designed. The effect of this system of reservoirs on past floods is shown in Table 1-7 and Plate Nos. 1-2 and 1-4. In the design of the Ansonia-Derby local protection project it is assumed that this system of reservoirs is complete and regulated as outlined in the Master Manual of Reservoir Regulation for the Housatonic River basin, dated June 1964.

9. STANDARD PROJECT FLOODS

a. Naugatuck River. The standard project flood developed for the Naugatuck River, as described in the "Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River," dated June 1958, will be used as criteria for establishing design grades for the walls and dikes on the main river. The standard project flood was based on the standard project storm rainfall, as described in Civil Engineer Bulletin No. 52-8, and unit hydrographs derived from analyses of record floods. The storm was oriented in the lower portion of the Naugatuck River basin to determine the maximum flows at Ansonia with the proposed system of reservoirs in operation. A mass rainfall curve and isohyetal map of the standard project storm and flood hydrographs at Ansonia are shown on Plate No. 1-5.

b. Beaver Brook. The standard project flood developed for Beaver Brook is described in the "Interim Report on Review of Survey, Ansonia-Derby Local Protection, Naugatuck River," dated 13 April 1960. The standard project storm rainfall for 10 square miles was applied to a

TABLE 1-7

EFFECT OF PROPOSED RESERVOIR SYSTEM IN REDUCING RECENT MAJOR FLOODS AND STANDARD PROJECT FLOOD - NAUGATUCK RIVER AT MAPLE STREET AND DIVISION STREET, ANSONIA, CONNECTICUT
(Drainage Area - 306 square miles)

	Discharge		Stage				
			Maple Street		Division Street**		
Flood	Natural (cfs)	Modified* (cfs)	Natural (msl)	Modified (msl)	Natural (msl)	Modified (msl)	Tide
Sept 1938	30,000	15,000 (est)	30.0	25.8	21.3	19.6	Abnormal
Dec 1948	32,700	15,600	30.3	25.9	18.8	17.4	Normal
Aug 1955	112,000	54,000	39.6	33.6	27.9	22.6	Normal
Oct 1955	40,000	23,000 (est)	31.5	28.3	23.8	21.4	Abnormal
SPF	148,000	75,000	42.7	36.1	33.0 (est)	30.0	Abnormal

* The proposed system includes the following reservoirs:
Thomaston, Hall Meadow Brook, East Branch, Northfield Brook, Black Rock, Hancock Brook, Hop Brook

** Based on estimated tailwater conditions of the Housatonic River at Shelton unit hydrograph derived from analyses of record floods. A graphical description of the Beaver Brook standard project flood is shown on Plate No. 1-5.

10. PROJECT DESIGN FLOODS

a. Naugatuck River. The protective works on the Naugatuck River will be designed for the standard project flood concurrent with an abnormal tide in Long Island Sound. As noted in Table 1-6, abnormally high tides have occurred concurrently with floodflows in the Naugatuck and Housatonic Rivers. It is, therefore, considered reasonable to assume that the maximum stage in the Ansonia-Derby area would occur: (1) at the time of the peak discharge in the Naugatuck River resulting from the standard project storm centered over that basin; (2) with a concurrent floodflow in the Housatonic River from the same storm; and (3) with an abnormal tide in Long Island Sound. A peak discharge of 75,000 cfs in the Naugatuck River concurrent with a flow of 145,000 cfs in the Housatonic River and an abnormal tide results in an assumed water surface elevation of 29 feet msl in the Naugatuck River at the downstream end of the protective works near Division Street bridge.

b. Beaver Brook. Maximum stages in Beaver Brook result from various combinations of discharges in the brook and concurrent stages in the Naugatuck River. The highest stages in the lower reach, between the mouth and Station 160+00, results from the stage of 32.0 feet in the Naugatuck River, produced by the standard project flood, with the concurrent discharge of 1,000 cfs in Beaver Brook. Upstream of Station 160+00 the maximum stage is produced entirely by the peak discharge in Beaver Brook. The peak discharge of the Beaver Brook standard project flood is 3,180 cfs.

A factory building spans the brook just downstream of Jewett Street bridge, limiting the normal channel capacity to about 1,500 cfs. With surcharge of the opening under the building to the level of overflow into Beaver Street, a maximum flow of 2,500 cfs can be discharged. In August and October 1955 total storm rainfall in the amounts of 7 and 9 inches were recorded at Ansonia, respectively, and major floods resulted. It is estimated that the peak flows in Beaver Brook were approximately equal and ranged from 1,000 to 1,200 cfs. Only minor damage resulted to the factory building. Enlargement of the channel under the building to satisfy the estimated standard project flood is considered impractical, hence, a discharge of 2,500 cfs, approximately 80 percent of the standard project flood and twice the flood of record, has been selected as the design flood for the Beaver Brook project. It is estimated that 2,500 cfs has a 0.25 percent chance of occurrence in any one year.

11. MODEL STUDY

It is proposed to make a hydraulic model study of the proposed protection measures and improvements along the Naugatuck River to assure adequate and economic design. A model study is considered necessary due to the complex flow conditions resulting from the combination of abrupt changes in channel section, skewed bridges and bends for which hydraulic losses cannot be reliably computed. Selection of grades for the dikes and floodwalls will be made after completion of the model study.

PART II - INTERIOR DRAINAGE

12. GENERAL

The proposed layout of the local protection project shown on Plate Nos. 1-9 through 1-12 is the result of hydrometeorologic and economic analyses of the Naugatuck River in the towns of Ansonia and Derby, Connecticut. Protection along the east bank will extend from the hydroelectric plant of the American Brass Company southerly to the confluence of Beaver Brook. The protection on the west bank is divided into two sections: (1) from the Ansonia Manufacturing Company to a point just upstream of Maple Street bridge; and (2) from the NYNH&H railroad bridge southerly to a point about 2,000 feet downstream of Division Street bridge. The proposed system of dikes and walls will intercept runoff from approximately 880 acres of interior area. Land uses in the interior basin are about 60 percent residential, 20 percent commercial and 20 percent industrial. Developments in the higher elevations of the drainage basin are largely residential. The high-value industrial and commercial developments are located on the narrow flat flood plain adjacent to the river.

13. TOPOGRAPHY

The sides of the river valley slope steeply to the flood plain which has a gentle north to south inclination. The flat flood plain comprises approximately 40 percent of the total drainage area. Roads slope towards the river with grades as steep as 15 percent.

14. INTERIOR DRAINAGE BASIN

The interior drainage basin consists of all contributing areas from which runoff would be impounded by the proposed dikes and floodwalls. The total basin has been divided into 10 areas (see Plate No. 1-8). Areas 1, 3, 6, 8, 9 and 10 are entirely above the project design flood stage. The runoff from these areas will be intercepted in drop inlets and carried by pressure conduits to the river, thereby, substantially reducing the pumping requirements. Areas 2, 4, 5 and 7 are considered low areas and are all contiguous to the protection. Drainage will be collected in these areas and discharged through the dikes or walls at the various pumping stations by gravity or pumping, depending on the concurrent river stage.

Information on the existing municipal drainage system is very sparse. The city drainage plans were destroyed in the 1955 flood with many other city records and no survey was made to replace them. Field observations indicate that, for the most part, the system is incapable of handling any significant amount of runoff. At the present time flows in the upper reaches of the basin are intercepted by streets running parallel to the river. Runoff not intercepted by inlets on these relatively flat streets flows to the intersections with the steeper streets which slope toward the river. These latter streets conduct the surface flow at considerable velocity to the flood plain.

15. HYDROLOGIC ANALYSIS

a. River stage and discharge frequencies. Modified stage frequency curves for the Naugatuck River at selected locations are shown on Plate No. 1-6. During the evacuation of stored flood runoff in the upstream reservoirs, the discharge in the river at Ansonia-Derby will be maintained at about 9,000 cfs for sustained period of time. Discharge frequency curves, modified by upstream reservoirs, indicate that a discharge of 9,000 cfs will have an average frequency of occurrence of once in three years. To minimize the duration of pumping, the design sump levels for pump activation should be established at or above the river stage produced by this flow of 9,000 cfs.

b. Coincidence of interior runoff and river stage. Plate No. 1-7 shows the rainfall rates in the Ansonia area concurrent with high river stages in the Naugatuck River during past floods, namely: August and October 1955, December 1948 and September 1938. During all four floods it would have been necessary to activate the pumps had the local protection project been in existence. The maximum hourly rainfall rates during the period of high river stage were 0.67 inch in 1938 flood, 0.20 inch in 1948 flood and just over 1 inch in both the October and August floods of 1955. As previously discussed the Naugatuck River is very flashy and flood stages develop within a few hours after the beginning of rainfall. In the future, floods in the Naugatuck River will originate in the uncontrolled areas below the flood control dams and will develop even more quickly at Ansonia-Derby than in the past. On the basis of flow duration information and the use of the procedure outlined in EM 1110-2-1410, the recommended storm for the design of pumping stations should have a 0.3 percent chance of occurrence. From U. S. Weather Bureau Technical Paper No. 40, such a storm in Ansonia would have a rainfall rate of approximately 0.7 inch per hour. However, in 30 years of record the area has twice experienced rainfall rates of 1.0 inch per hour coincident with high river stage. Considering experienced conditions and the flashy nature of the river, a design storm of 1.5 inches per hour, or 50 percent greater than the maximum experienced was adopted. Because of the high value of the flood plain lands, resulting from intensive industrial and commercial development, it is considered infeasible to reserve areas for temporary storage of interior runoff.

16. DESIGN CRITERIA

The criteria used in the design of the pressure conduits, interceptor storm drains, pump stations and subdrains are as follows:

a. Pressure conduits. Pressure conduit systems shall be designed to intercept and carry a 100-year storm runoff.

b. Interceptor drains. Interceptor drains along the line of protection collect runoff that would normally flow overland to the river. They will be designed to carry a 10-year storm with the drain running full and discharging by gravity.

c. Pump stations. Pump stations will be designed to discharge the runoff produced by a rainfall rate of 1.5 inches per hour against the design river stage.

(1) Where pumping stations are built into floodwalls, the pump discharge line will pass through the wall and the pumps will discharge against the head of water in the river. Where pumping stations are built alongside dikes, the pump discharge line will pass over the top of the dike.

(2) Separate gravity outfalls will be provided at each pumping station and these will be designed for the 100-year storm runoff.

d. Underdrains. Underdrains will be designed to carry flows based on an infiltration rate of .025 gpm per foot of differential head per foot of dike.

e. Sluice gates. Sluice gates will be installed on all pressure and gravity discharge conduits that pass through the line of protection. The gates will be located on the riverside of the line of protection and will permit emergency closure in the event of conduit failure. Flap gates will also be installed on the outlet ends of all conduits.

17. DAMAGE POTENTIAL IN PROTECTED AREAS

The existing industrial and commercial developments on the flood plain are presently subject to short duration flooding from high intensity rainfall due to rapid concentration of runoff from the steep slopes accumulating on the flood plain. The damage potential cannot be determined precisely because the complex overland flow pattern and the head-discharge relationships of the existing drainage structures may be substantially affected by ice and other debris during critical flood periods. Also the effectiveness of future drainage improvements by the city could improve the situation markedly.

The proposed drainage systems to be constructed with this project will prevent any increase in the flood potential of the low lying industrial and commercial areas by intercepting a large portion of the runoff from the upper slopes and by providing drainage along the lines of protection to prevent impoundment. However, the existing condition of infrequent shallow flooding in the streets and on the highly industrial and commercialized flood plain during short duration, high intensity rainfall will not be completely eliminated unless the city makes significant improvements in the interior drainage system. It is estimated that with the proposed drainage system, during maximum 100 year rainfall intensities, depth of flow in streets and other natural waterways might reach depths of 6 inches. Depths of ponding in depressions over present drainage inlets might reach 1 to 1.5 feet. This flooding would be of relatively short duration, however, probably not exceeding periods of more than 1 to 2 hours. In the highly commercial and industrial areas such flooding is presently experienced and would result in some inconvenience but due to its short duration, it is tolerated with little permanent damage. This level of ponding is considered to be within the limits of ponding level "C" as discussed in EM 1110-2-1410.

Because of the flat topography of the flood plain, depths of flooding from the standard project storm would not be excessive and would be of relatively short duration, varying from 2 to 3 hours. Therefore, there would be little threat to human life, losses would not be catastrophic and ponding depths would be within the limits of stage "D" as described in EM 1110-2-1410.

18. DRAINAGE SYSTEMS

Location, topography and drainage of the 10 interior drainage areas are discussed in the following paragraphs. A proposed drainage system that will meet the prescribed criteria is presented for each of the 10 areas. Design discharges for all areas are shown on Table 1-8.

a. Drainage Area 1. Area 1 is located northeast of the main business district of Ansonia on the east side of the Naugatuck River at the upstream end of the protective works. The area is typical of the steep, medium concentrated, residential areas of the city with some scattered small business establishments and the northern portion of the American Brass Company yard. The watershed, consisting of 69 acres, is approximately 3,000 feet in length with a differential in elevation from 50 to 300 feet msl. The pattern of overland flow is determined by the area's network of streets, generally running transverse to the slope. Flows exceeding the limited capacity of the existing storm sewers concentrate on Third Street. In order to contain and direct these flows to the main collection point at the west end of the street, a roadway inlet with grating will be installed across Liberty Street to prevent the runoff from being diverted southward at this point. The water collected in this inlet will be carried to the west end of Third Street in a 36-inch storm drain. Large inlets will be installed at this point to insure complete collection of existing city drains, the new 36-inch storm drain and the remainder of the surface runoff. At the present time surface flows which collect at this location flow down a chute into a 42-inch gravity drain located in the American Brass Company parking lot which discharges into the Naugatuck River.

Drainage from Area 1 concentrating at the west end of Fourth Street, where the gate to the American Brass Company parking lot is located, will be picked up by the existing drains and discharged into the headrace canal.

All drainage will be intercepted before it gets to the lower areas of the American Brass and Farrel Corporation yards. This will considerably reduce pumping requirements at the Maple Street pumping station. The runoff will be conducted to the river by existing drains and a new pressure conduit. The total drainage design runoff for the area is 160 cfs. This will be divided into a flow of 30 cfs which goes into the process water canal through city drains and overland flow and a 130 cfs flow into the new drainage system shown on Plate No. 1-9. The existing 42-inch gravity drain under the American Brass Company parking lot from a point off the end of Third Street to the vicinity of the screen house will be supplemented with a new 42-inch pressure drain. A new manhole will be constructed west of the screen house where the existing and new drains connect. From this manhole a new pressure conduit, varying in size from 48 to 60 inches, will be laid to the hydroelectric plant.

The elevation of the water in the process water canal or headrace where it is proposed to discharge 30 cfs during the 100-year storm can be controlled by the normal operation of the existing weir at the hydroelectric plant. The headrace canal will be protected from the 100-year stormflows coming from the reservoir to the north by a concrete wall barricade installed across the canal. It is anticipated that these flows will be 4 feet higher than normal flow elevation which is slightly higher than the top of the overflow weirs at the hydroelectric plant.

The barricade will have a 3' x 3' opening positioned to permit an adequate passage for the American Brass Company's process water requirements during low flows. During the 100-year storm, the additional head on this opening will considerably increase the flow into the headrace, but the existing overflow weir south of the barricade will be capable of discharging the excess flow into the river. A slide gate will be installed to close the opening completely if there is a breach in the bank of the headrace.

The 100-year stormflows have been predicated on a low river stage at the discharge points. Conditions with the river at the project design flood elevations outside the protection and concurrent rainfall have been considered. As stated above rainfall occurring with the river at flood crest will be 1.5 inches per hour for design purposes. The total flow in Area 1 with this storm will be 50 cfs. This flow can be handled easily by the system designed for the 100-year storm.

b. Drainage Area 2. This area is representative of the very flat narrow flood plain bordering the river through the city. Area 2 is bounded by the Naugatuck River, Liberty, North Main and Maple Streets. It comprises the plant areas of the American Brass and Farrel Companies upstream of Maple Street and totals 43 acres. The area is highly industrialized with almost complete coverage by buildings or pavement. It is long and narrow in shape with a relatively flat pitch towards the river and Maple Street. The maximum flow path is approximately 3,000 feet. The area is underlaid with an extensive system of storm and building drains and two tailraces. Some of the individual storm drains discharge directly into the river. The rest discharge into the tailraces which also carry process water wastes.

Drainage for this area will be provided by installing an intercepting drain along the floodwall and through the company yards which will conduct both storm runoff and process water to the Maple Street pumping station located south of the Maple Street bridge. This drain is designed for a 10-year storm with a cumulative flow of 125 cfs. In addition to storm drain runoff, subdrains and city water wastes also contribute to the system. Flow will discharge by gravity during normal river stages but will be pumped during high river stages. The pump activation level is predicated on criteria outlined in paragraph 15. The upstream end of the new collector drain begins with interception of the first major outfall pipe, an 18-inch line that drains the American Brass Company parking lot. This line, as indicated in the discussion of Drainage Area 1, will have most of the upstream inlets contributing to it diverted to the pressure conduit for Drainage Area 1. Contributions to the line under the new layout will include some inlets located on the low area of the American Brass Company yard. The discharge line from this manhole will be a 24-inch drain carrying 12 cfs, 360± feet southerly to a new manhole over an existing 18-inch mill drain currently discharging into the river.

There are three other major outfalls to the river south of this location and each will be terminated with a manhole and diverted into the new interceptor. Contributions from these sources include the storm drainage, process water wastes and subdrains. A change in alignment will occur in the drain where it intercepts the existing tailrace in the American Brass Company yard. A special manhole will be constructed at this point. The discharge line from this manhole will be a 42-inch drain carrying 58 cfs. The next manhole will have to be positioned to permit the drain line to clear the footings of the existing viaduct and to minimize the trench excavation across an open storage shed with a concrete floor. The next manhole will also be used for an alignment change directing the drain in a southerly direction in the railroad yard. An intermediate manhole will be located 350 feet south of this manhole. A new manhole will be located 350 feet further south over an existing 20-inch drain line that handles plant waste and municipal storm drainage from the streets east of the Farrel Corporation yard. The flow in the interceptor will increase to 72 cfs in the 48-inch effluent drain from this manhole. The next outfall to be picked up by the interceptor will be the tailrace under the Farrel Corporation yard. This will require a special structure and the design discharge of 120 cfs from this point in Area 2 would be contained in the 54-inch discharge line from this structure. A manhole south of the Maple Street bridge will be used for an aligmment change into the pumping station. This structure will have a grate to collect all local drainage and excessive overland flows from Area 2. The design discharge from this structure will be 125 cfs.

c. Drainage Area 3. This area is located northeast of the main business district of Ansonia on the east side of the river. The area, containing approximately 153 acres, is largely residential and similar in development to Area 1 except somewhat less concentrated. Most of the area is serviced by a municipal drainage system. The ground generally slopes towards the river and drops about 250 feet in a maximum distance of 1 mile. During periods of high rainfall intensities, runoff from the upper portion of the area collects into a natural draw which starts near Granite Terrace, continues downstream between Ellis Street and Rockwood Avenue, crossing the intersection of Woodridge and State Streets, and discharges onto a large flat area between Lock and Crowley Streets. Outflow emerging from this area presently converges with flows from the southerly sections of the drainage area at the intersection of State and North Cliff Streets. A trunk line storm sewer, located in the bottom of the aforementioned draw, continues down State Street through the Farrel Plant and discharges into the river. Extensive roadway gratings and gutter inlets will be installed at the intersection of State and North Cliff Streets to positively intercept all overland flow and prevent it from going down Main Street into Area 4. Total flow from the area for the 100-year storm will be 275 cfs. Of this quantity, it is estimated that 250 cfs will be overland flow and have to be picked up in the area indicated above.

The new roadway grating and gutter inlets on State Street will be discharged into a 60-inch pipe running down State Street to a new inlet structure on North Main Street. From this point the 60-inch drain will turn southerly to an existing storm drainage structure that will be rebuilt to take the 60-inch drain. From this point the drain becomes a pressure conduit, enters the Farrel Corporation yard and follows the general alignment of the existing 20-inch drain. At a point west of the railroad tracks between the Farrel Corporation buildings, another manhole will be added to provide an alignment change directing the 60-inch conduit in a southerly direction, roughly paralleling the new gravity system, to a point south of the Maple Street bridge. Here another manhole will change the alignment to direct the drain westerly to the river. The discharge line will pass through the floodwall just north of the Maple Street pumping station. Existing city storm drains will be used whenever possible to supplement the new system. The 20-inch discharge line will be tied into the manhole on the gravity system at the railroad tracks if feasible and the remainder of the line will be abandoned.

All manholes on the pressurized section of the line will be pressure manholes with secured and gasketed frames and covers capable of withstanding the anticipated uplift at high river stage. The conduit will have a sluice gate at the floodwall and a flap gate on the end of the pipe.

With the river at high stage the water elevation at the discharge end of the pressure conduit will be elevation 37.0±. The grate elevations at the beginning of the pressure conduit elevation are 60.0±. These elevations provide a head more than adequate to discharge the 117 cfs of the design storm during high river stages.

d. Drainage Area 4. The watershed of Area 4 is located on the east bank of the Naugatuck River between Maple Street and Beaver Brook. It includes the main business district, a downstream residential area which is in the process of being cleared for urban renewal, and some of the adjacent inland hills which are largely residential. The area consists of approximately 125 acres, with a major portion on the very flat low lying flood plain, similar in topography to Area 2. The maximum flow path is approximately 4,000 feet and drops about 100 feet, with most of the change in elevation occurring in the upper reaches of the area. The entire area is presently serviced by a system of municipal storm drains which discharge into the river and Beaver Brook. The area was divided into subareas to establish pipe sizes for the interceptor drain. It is anticipated that the utility alignments for the southern part of the area being leveled in preparation for an urban renewal project will be changed radically. There are no plans showing the proposed layout available at this time. The drainage layout will be coordinated with the interceptor drain and the pump house when they are made available.

The upstream end of the interceptor drain for Area 4 will begin with an existing catch basin in the municipal parking lot off the west end of Bank Street. It will intercept a storm drain from Bank Street and the subdrain piping between the Maple Street pump station and the Bridge Street bridge. A 24-inch drain will carry the 27 cfs from this manhole approximately 250 feet south to the next manhole that will be located over an existing drain in an abandoned tailrace. The condition of the tailrace is not known. It is assumed the only flows into it, if any, are from building drains. The manhole over the tailrace will also be utilized for an alignment change. The interceptor drain in the reach from Bank Street to Bridge Street will parallel West Main Street to avoid having to go through the bridge abutment. The manhole over the tailrace and the new alignment manhole 300 feet south will pick up the existing catch basins in the municipal parking lot. The discharge line from this structure will be a 30-inch pipe with a flow of 38 cfs and it will be tied into the existing storm drain manhole at the intersection of Bridge and West Main Streets. The existing 42-inch discharge pipe from this manhole currently runs parallel to the south side of the East Bridge Street bridge abutment and discharges into the river. The section of the existing drain from the aforementioned intersection to a new manhole on the line just east of the new floodwall will be incorporated as part of the interceptor drain. From this manhole the interceptor drain will take a southerly course and the existing 42-inch drain from this manhole to the point of discharge in the river will be abandoned. The flow in the existing 42-inch drain for this reach will be 75 cfs. In order for the pipe to carry this flow, the manhole at the intersection of Bridge and West Main Streets will have to be surcharged less than 1 foot. The existing system is deep enough at all points to avoid ponding at inlets. The discharge pipe from the new manhole constructed over the existing 42-inch drain will be a 48-inch pipe that will follow a southerly course through several manholes provided for alignment changes to the point where it intercepts the existing 36-inch drain from the American Brass Company's southern plant. The flow from this manhole will be 106 cfs and will be carried to the next manhole off the end of Cheever Street with a 54-inch pipe.

An existing 24-inch tile drain off the end of Cheever Street is assumed to be the discharge line for a system of municipal storm drains. A new manhole will be constructed over this line connecting to the interceptor manhole on the 36-inch drain from American Brass Company. The flow discharging from this manhole over the Cheever Street drain will be 162 cfs and will be carried southerly in a 60-inch pipe to the Front Street pumping station. All surface run-off and drainage picked up in the city system south of this point will be carried to or picked up at the pumping station. It will amount to 58 cfs for the 10-year design storm used for the interceptor, therefore, the total gravity discharge of a 10-year storm at the Front Street pumping station will be 220 cfs.

All new drain manholes on the interceptor line close to the protection will have grates and the areas in the immediate vicinity of these manholes will be graded to divert local surface runoff into them.

In the event of a 100-year storm, field observations indicate that there is not enough inlet capacity in the city storm drainage system to prevent considerable overland flow. The general topography for Area 4 shows a fairly uniform and flat slope running from north to south, therefore, any runoff that is not picked up by the city storm drains or the new interceptor drain will wind up in the low area in the vicinity of the Front Street pumping station. Roadway gratings with large inlet capacities will be provided in this area to obviate ponding and to discharge overland flows along with those in the interceptor into the gravity discharge channel bypassing the pumping station. This flow will be 315 cfs and the gravity discharge line through the protection will be sized to carry it.

During the project flood with the river at high stage, the existing storm drainage system in conjunction with the interceptor drain will carry the flows from the design storm (1.5 inches per hour) to the Front Street pumping station where the pumps have been designed to handle the design flow of 130 cfs.

e. Drainage Area 5. Area 5 is located on the west bank of the Naugatuck River and consists of the grounds of the Ansonia Manufacturing Company located in the flood plain and an area to the south which would contribute runoff to the Ansonia Manufacturing area via Maple and River Streets. Maple Street consists primarily of commercial establishments and residences while River Street is an access road to the Ansonia Manufacturing and American Brass Companies. There is an existing system of municipal drains along the two streets and the Ansonia plant also has a system of inlets on which there is little information available.

A new storm drain interceptor will tie into the existing system in the vicinity of the bend on River Street. The drainage collected at this point will include the runoff from the Westside Shopping Center and Maple Street. The existing storm drain on Maple Street in the vicinity of the proposed flood gate will be plugged. The water will be permitted to back up in the existing drain from this point to the storm drain manhole at the intersection of Maple and River Streets. From this manhole it will flow down to the bend in River Street in the existing storm drain and from there, into the new 18-inch interceptor beginning at this point. The new interceptor runs parallel to River Street collecting all curb inlets on the street and increasing in size to 36 inches where, from the last structure, it can be discharged to the river by gravity or diverted to the River Street pumping station on the north side of the Ansonia Manufacturing Company lot. All existing drains discharging to the river will be plugged and the flows diverted to the new interceptor.

During the 100-year storm both street gates will be open and all flows in excess of the capacity of the gravity bypass at the pumping station will flow overland through the street gates. The topography of Maple and River Streets in these areas provide a natural discharge conduit to the river for the 100-year runoff.

f. Drainage Area 6. Area 6 is a highly residential area located directly to the west of Area 5. It consists of approximately 42 acres with a maximum flow path of 1,500 feet and a drop of 40 feet. Existing storm sewers from this area discharge into an existing drainage chute on the steep bank to the west of the Ansonia Manufacturing grounds. Runoff in excess of the storm sewer capacity flows down Maple Street and into the river. The low point of the area is in the vicinity of the intersection of Jackson and Maple Streets. At this point a drainage structure consisting of a reinforced concrete trench with a roadway grating extending from curb on Maple Street will intercept all flows heading toward Area 5. This structure will pick up all overland flows from the 100-year storm in addition to all flows from the existing storm drain lines that converge at this point. The structure will discharge into a new drainage chute. The discharge at this chute is in the unprotected area between Maple Street and the new floodwall on the west side of the Ansonia Manufacturing Company property.

A 100-year storm will contribute 120 cfs to the new structure and it will be discharged into the new chute with a 48-inch pipe. This design will also eliminate an additional 30 cfs pumping requirement at the River Street pumping station during a 1.5 inch per hour storm concurrent with high river stage.

g. Drainage Area 7. This area is located on the west bank of the Naugatuck River between the NYNH&H railroad bridge and the tie-back dike in Derby. It includes land in both the cities of Ansonia and Derby. The area is divided into two topographically different sections. The low area between the NYNH&H railroad tracks and the line of protection is comparatively flat with a maximum difference of 20 feet in elevation. This section is largely undeveloped except for a few industrial establishments located adjacent to the railroad and an open air theater. The upper section, west of the railroad tracks, is on a side hill with elevations varying from 40 feet to 160 feet msl. The higher section slopes gently to moderately toward the river becoming very steep along the border of the flood plain area. The total area covers 180 acres and the maximum distance traveled by runoff in the area is about 4,500 feet. The higher area west of the railroad is mostly residential with a few commercial establishments. Much of the high western area is drained by storm sewers or diverted by the railroad embankment to a 65 x 40 inch drain located adjacent to Division Street, between Mill Street and the river.

It is proposed to construct a pumping station and gravity outfall near the end of the existing drain near Division Street bridge. A drainage line with surface inlets will be installed along the toe of the dike north and south from this location to intercept the remaining drainage from the low lying flood plain area. The northerly drain will also have sufficient capacity to conduct effluent from a future sewage plant during high river stage.

The interceptor drain along the dike north of Division Street begins with a 24-inch storm drain and culminates with a 60-inch drain discharging into the junction structure "Special Manhole #3". The flow in the 60-inch drain will be 118 cfs for the 10-year storm. This area at the present time is being used as the town dump. It is the proposed site for a future sewage treatment plant. The perimeter of the drainage area consists of Mill and Division Streets and the dike from the railroad bridge south. The area to the west of Mill Street except as noted hereinafter, is picked up in municipal drains and is conducted to "Special Manhole #3" by the existing 65 x 40 inch pipe arch along Division Street.

The interceptor drain along the dike south of Division Street begins by picking up an existing 24-inch drain south of the golf putting course and terminates with a 48-inch drain discharging into "Special Manhole #3". The flow in this drain will be 57 cfs for the 10-year storm. At the present time the area consists of the grounds and buildings of the Charlton Press Company railroad tracks and an outdoor drive-in theatre. The perimeter of the drainage area consists of Mill and Division Streets and the dike south of Division Street.

In the design of the interceptor drains along the dike it was assumed the areas will eventually be filled and uniformly graded. For design purposes the area was subdivided into smaller drainage areas and pipe sizes predicated on the contribution from each. It is anticipated that additional inlet capacity will be added along with future development. The southerly part of Area 7 between Mill Street and Area 8 will be discharged in two ways:

(1) An existing drain beginning in the vicinity of the Mill and Division Street intersection running south and terminating with the existing 36-inch drain under Route 8 will handle all storm drain runoff when the river elevation is less than the invert of the 36-inch pipe where it discharges just south of Route 8. At the present time the existing line in the vicinity of the aforementioned intersection is plugged.

(2) When the river rises above the discharge point of the existing 36-inch drain under Route 8, the drain will be closed with a sluice gate and storm drainage will back up in the line to the point where it is presently plugged. At this point the existing drain will be unplugged and reconnected to the drainage structure that leads to the 65 x 40 inch pipe arch. This connection will be made with an overflow weir that will permit the backed up water in the 24-inch drain to get into the pipe arch and from there to the pump station for discharge to the river. The overflow weir will prevent normal flows in the pipe arch system from getting into the existing 24-inch drain along Mill Street.

h. Drainage Area 8. Area 8 contains approximately 60 acres and is located adjacent to Route 8 just to the west of Area 7. Presently, runoff from this area is collected in a natural steep channel which is intercepted at the foot of the slope by a 36-inch conduit under Route 8. During the standard project flood, backwater would occur in this conduit from the river to within the protected area.

It is proposed to install sluice and flap gates on the 36-inch gravity line and construct a 48-inch pressure conduit with the inlet above the design flood level to discharge the runoff from Area 8 directly into the river. This conduit will carry a flow of 140 cfs for a 100-year storm. A sluice gate structure will be installed on the line where it goes through the dike and a flap gate added to the end of the line. Diking and grading will be required in the vicinity of the inlet end to divert surface flows from Area 8 to the conduit. All manholes on the pressure conduit will be pressure manholes with secured and gasketed frames and covers capable of withstanding the anticipated uplift at high river stage.

i. Drainage Area 9. Area 9 consists of approximately 90 acres of steep residential area to the east of Area 4. Beaver Brook roughly parallels its east side and serves as a drainage disposal facility for some of the storm drain and surface flows. The total runoff during a 100-year storm has been computed as 250 cfs. It is estimated that 100 cfs of this total does not go into Beaver Brook either by storm drains or surface flows. Therefore, the 100 cfs flows to the low point of the area i.e. the intersection of Beaver and Central Streets. A drainage structure with a roadway grating extending from the curb on one side to the paved parking area on the other will be constructed on the south end of Beaver Street and the drainage therefrom will be conducted in a 48-inch drain across Central Street and into Beaver Brook just south of the Central Street bridge over the brook. The bed of Beaver Brook is to be lowered in this area to provide adequate head for discharge of the new street inlet into the brook. With the water in Beaver Brook at project flood elevation, there will be approximately 5 feet of head available to discharge storm drainage from the new structure. Diverting these flows from Area 4 will reduce pumping requirements at the Front Street pumping station.

j. Drainage Area 10. A large high hill, located on the east side and overlooking the city of Ansonia, comprises Area 10. The area contains approximately 105 acres, with a maximum flow distance of 5,300 feet. The area is a recently developed, medium density, residential section consisting of single family units, typical of suburban type housing. The natural slope of the area is toward Beaver Brook and drops from elevation 400 feet msl to 150 feet. The streets in the development divert the excessive surface runoff to Hill Street, which in turn conducts flows to Central Street.

Runoff which flows down Central Street will be prevented from entering Area 4 by the installation of an interceptor at Central Street bridge. This interceptor will consist of extensive grating on the bridge deck which will drop flows into Beaver Brook. The 100-year storm flow from this area will be 295 cfs. This positive interception also precludes the possibility of adding to the pumping requirements at the Front Street pumping station.

19. SUBDRAINS

The design of seepage flows is based on a flow rate of .025 gpm per foot of differential head per foot of dike or floodwall. This is an approximation based upon incomplete knowledge of the structures and is subject to modification in final design. Maximum subdrain flows will occur with the river at high stage. A 20-foot head on a subdrain would yield a flow of .0011 cfs per lineal foot. With an 8-inch CM subdrain pipe on a .005 slope, the subdrain could run for 550 feet before running full. The discharge of subdrains will always be into the new interceptor drains which are designed for a 10-year storm. The design storm with the river at high stage is considerably less than this, therefore, with maximum flows in the subdrains, the interceptor drains will still be running less than full.

The subdrains that are to be installed between the floodwall and the buildings from the Maple Street bridge to a point 1100 feet north on the east bank of the river shall be 12 inches in diameter. There are numerous small drains discharging to the river in this area and because of the lack of space between the buildings and the floodwalls, it is planned to use the subdrain as a combination drain to pick up these small flows.

TABLE 1-8

INTERIOR DRAINAGE DESIGN DISCHARGES
AND RATIONAL FORMULA DATA

Area No.	D.A. (acres)	Composite Infiltration Factor C	Time of Concentration T_c (minutes)	100-Year Rainfall Rate (inches/hr)	10-Year Rainfall Rate (inches/hr)	100-Year Q (cfs)	10-Year Q (cfs)	Q* Rainfall Rate 1.5"/hr. (cfs)
1**	69	0.5	30	4.6	3.2	160	110	50
2	43	0.9	30	4.6	3.2	180	125	60
3	153	0.5	45	3.6	2.4	275	185	115
4	125	0.7	45	3.6	2.4	315	220	130
5	13	0.9	15	6.4	4.4	75	50	20
6	42	0.5	20	5.6	3.9	120	80	30
7**	180	0.5	40	3.8	3.2	340	290	135
8	60	0.5	30	4.6	3.2	140	95	45
9	90	0.5	20	5.6	3.9	250	175	70
10	105	0.5	20	5.6	3.9	295	205	80

* This rate has not been modified for T_c

** See text for modification of this drainage area

CORPS OF ENGINEERS | U. S. ARMY

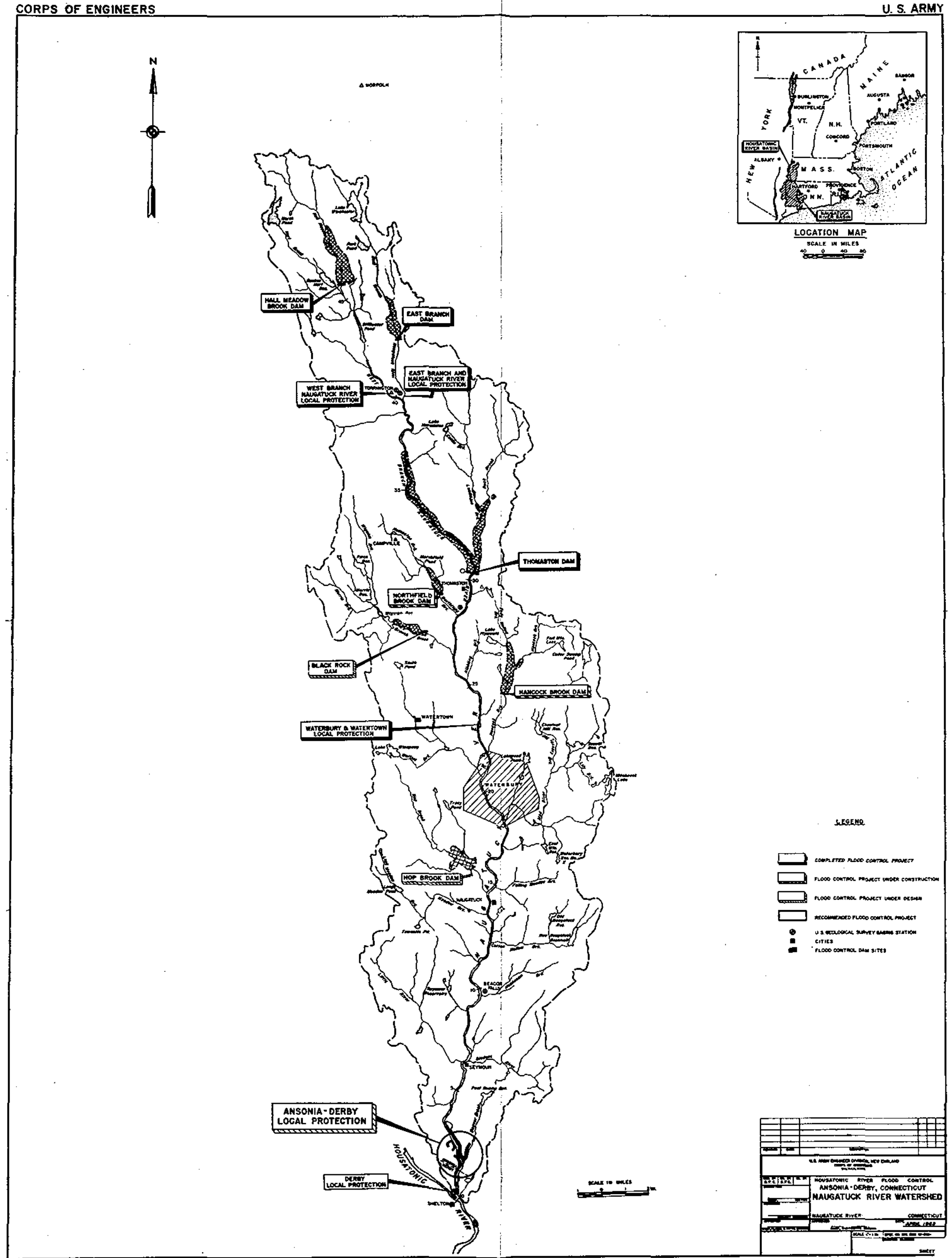
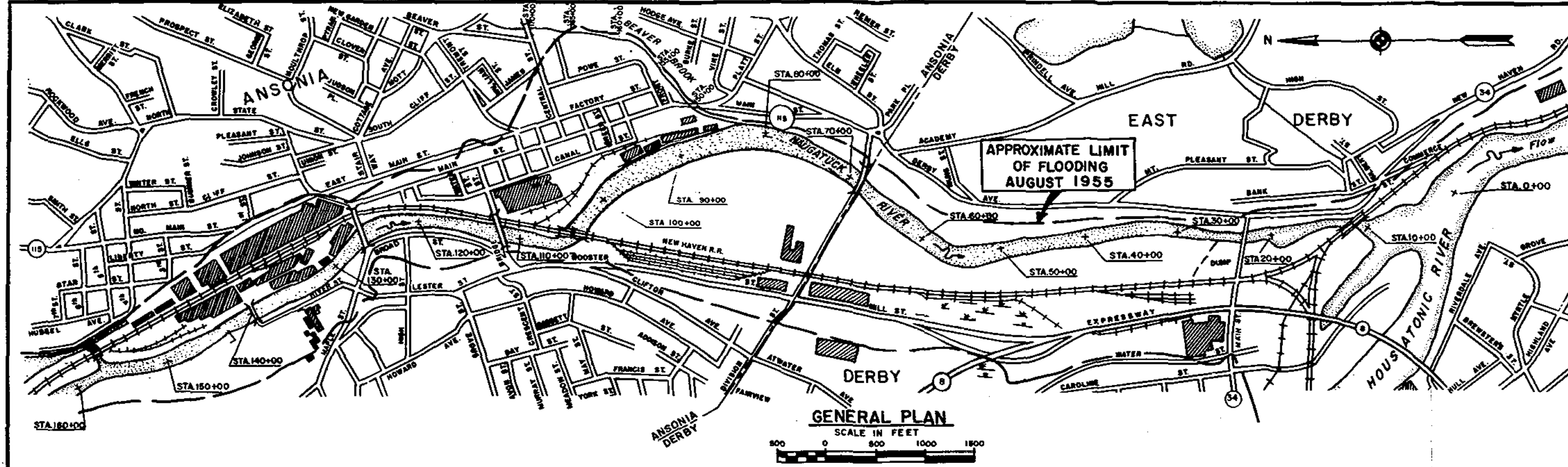
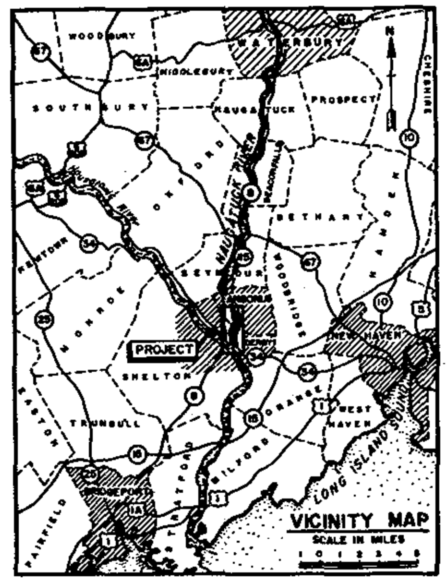


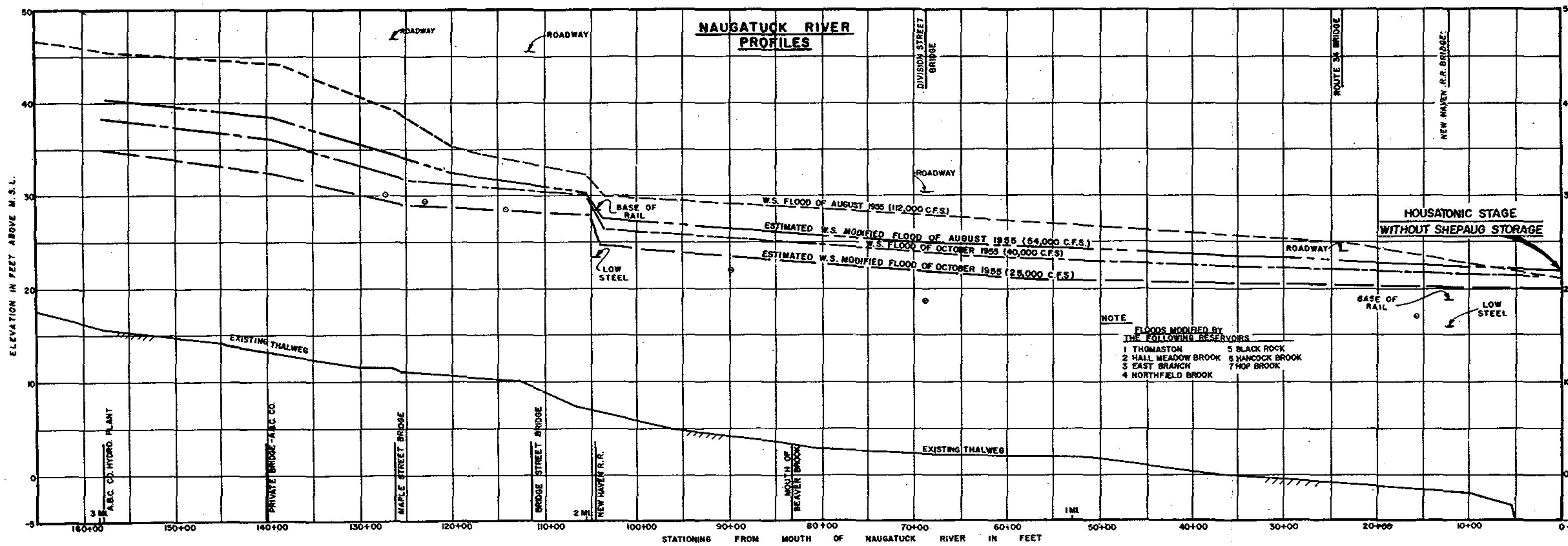
PLATE NO. 1

CORPS OF ENGINEERS

U. S. ARMY







LEGEND

- — — — FLOOD OF AUGUST 1955 PROFILE
- ——— FLOOD OF OCTOBER 1955 PROFILE
- ⊙ FLOOD OF DECEMBER 1948 HIGH WATER MARKS
- /////// THALWEG
- — —— MODIFIED OCTOBER 1955 FLOOD
- —··— MODIFIED AUGUST 1955 FLOOD

REVISION	DATE	DESCRIPTION	BY

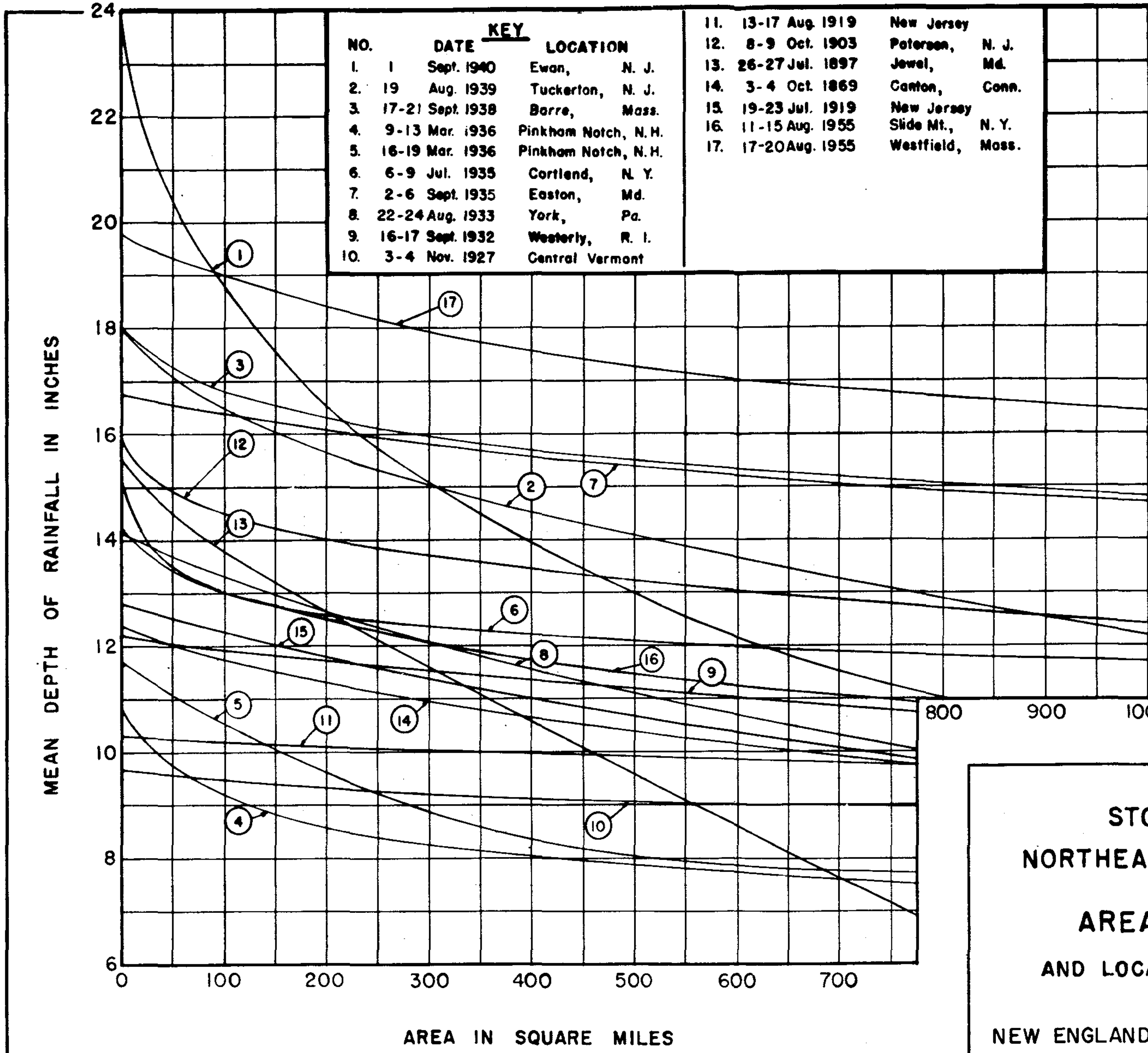
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

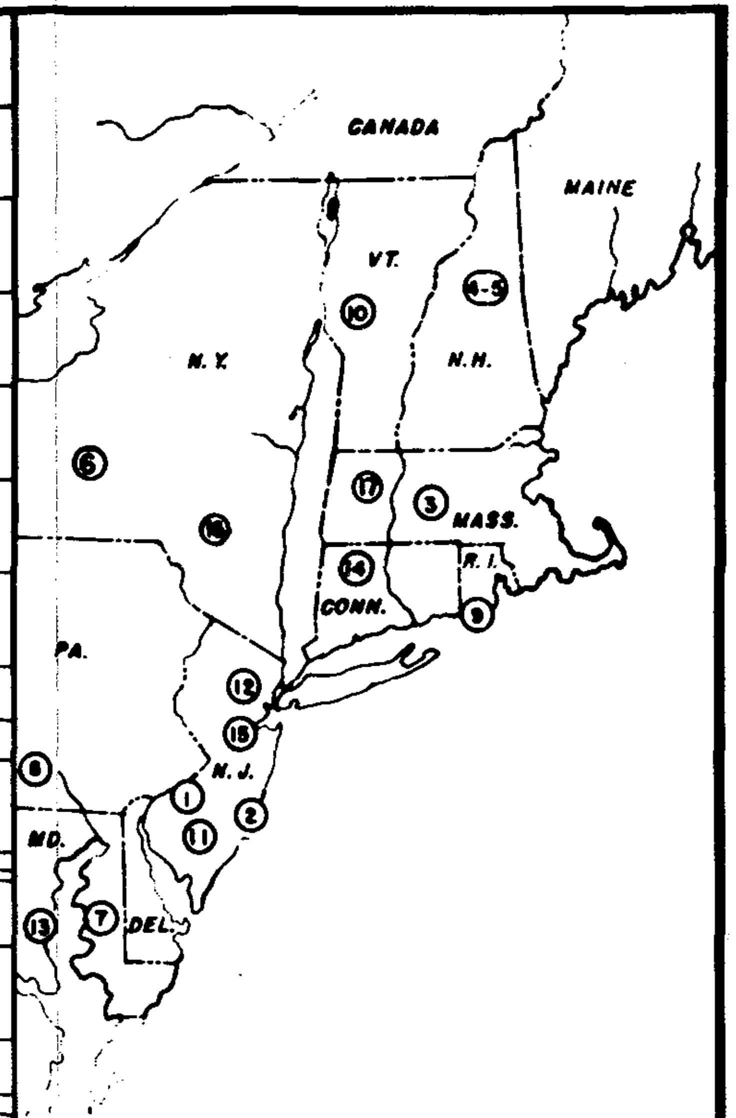
DR. BY | TR. BY | CK. BY

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA-DERBY, CONNECTICUT
LOCAL PROTECTION
PLAN & PROFILES
NAUGATUCK RIVER, CONN.

ENGINEER
SUBMITTED BY
CHIEF HYD. & HYD.
CHIEF, PLANS & SPECS BRANCH
APPROVED
CHIEF ENGINEERING DIVISION
DATE: APRIL 1965
SCALE: AS SHOWN
DRAWING NUMBER

PLATE NO. I-2



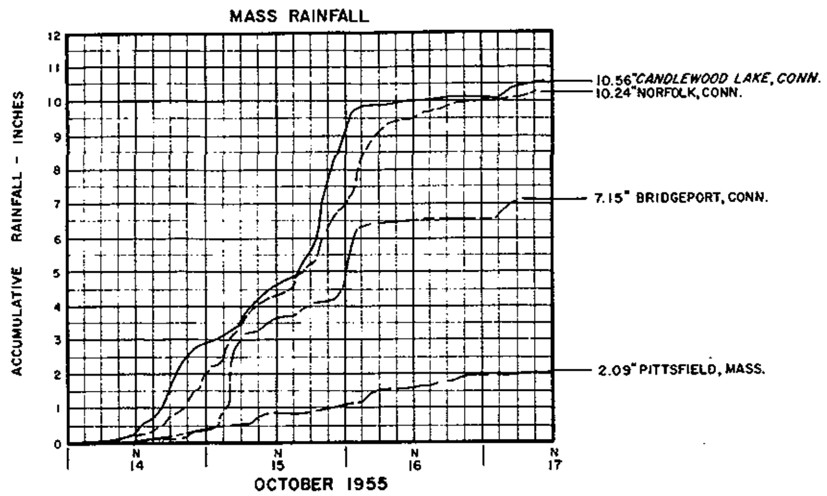


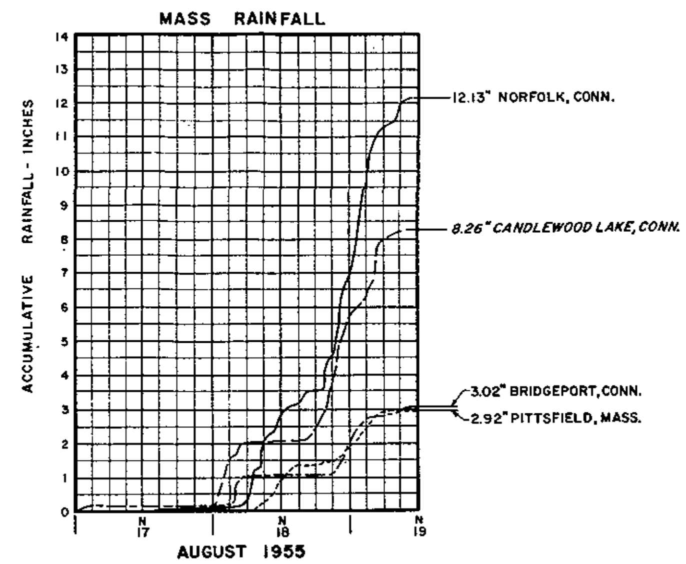
LOCATION OF STORM CENTERS

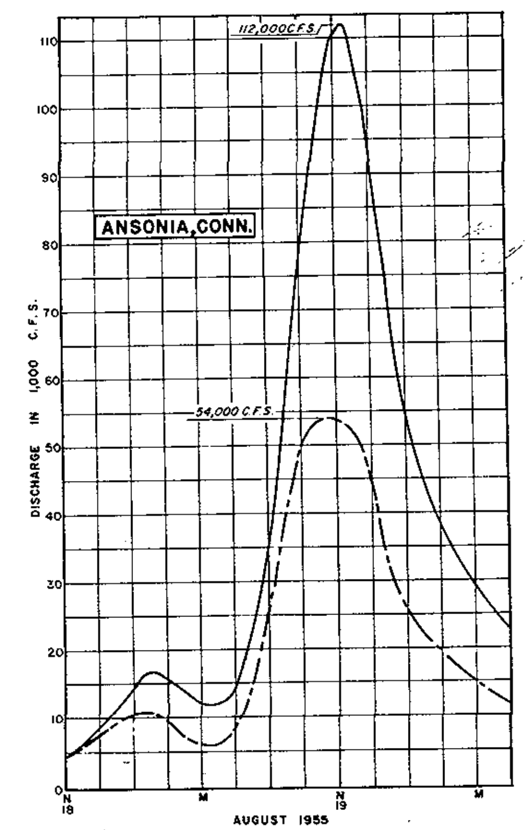
STORMS OF RECORD
NORTHEASTERN UNITED STATES
AREA - DEPTH CURVES
AND LOCATION OF STORM CENTERS

NEW ENGLAND DIVISION BOSTON, MASS
APRIL 1965

PLATE NO. I-3

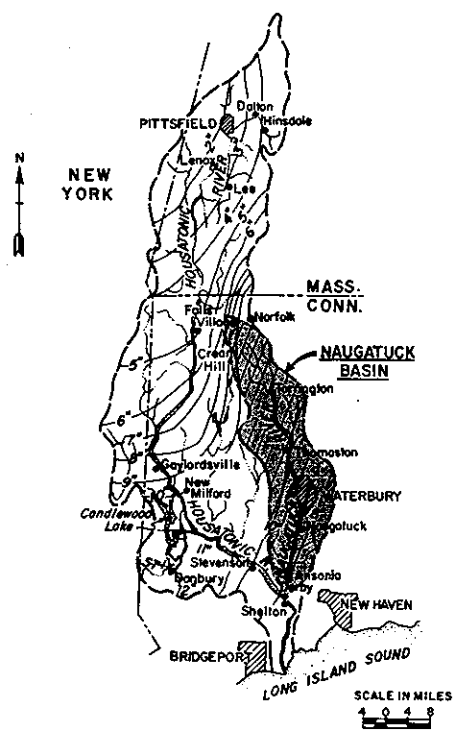




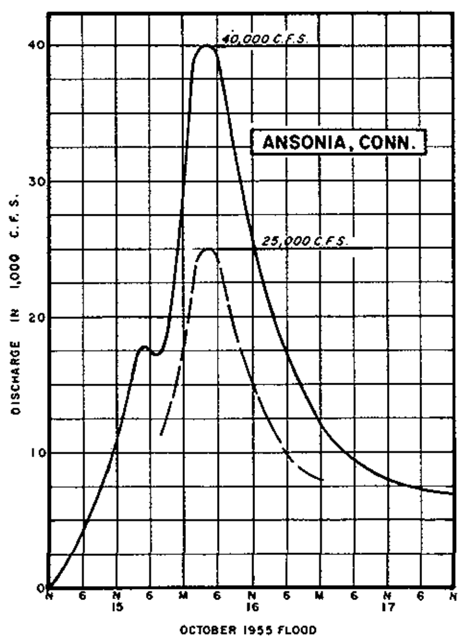


NATURAL

MODIFIED BY COMPREHENSIVE SYSTEM OF RESERVOIRS
THOMASTON RESERVOIR (PLUS HALL MEADOW BROOK AND EAST BRANCH RESERVOIRS)
NORTHFIELD BROOK RESERVOIR.
BLACK ROCK RESERVOIR.
HOP BROOK RESERVOIR.
HANCOCK BROOK RESERVOIR

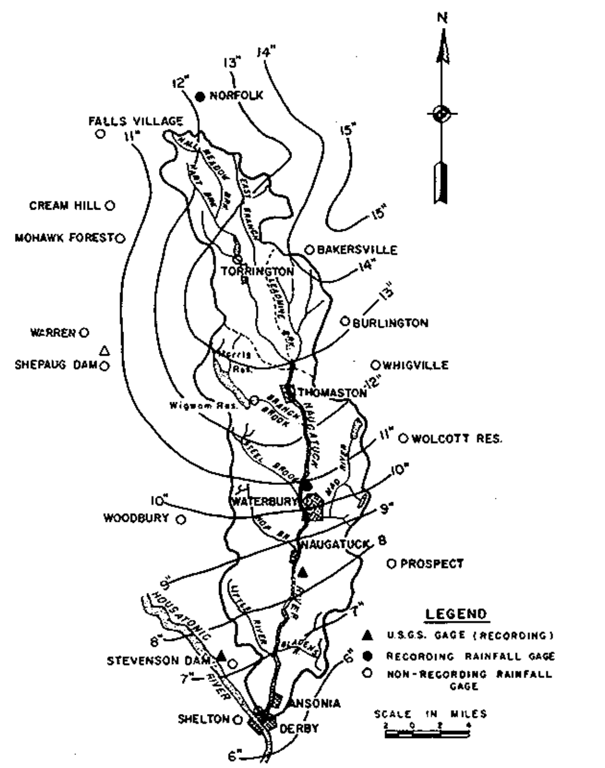


ISOHYETAL MAP
STORM OF
OCTOBER 14-17, 1955



NATURAL

MODIFIED BY COMPREHENSIVE SYSTEM OF RESERVOIRS.
THOMASTON RESERVOIR (PLUS HALL MEADOW BROOK AND EAST BRANCH RESERVOIRS)
NORTHFIELD BROOK RESERVOIR
BLACK ROCK RESERVOIR
HOP BROOK RESERVOIR
HANCOCK BROOK RESERVOIR



ISOHYETAL MAP
STORM OF AUGUST 17-19, 1955
NAUGATUCK RIVER BASIN

REVISION | DATE | DESCRIPTION | BY

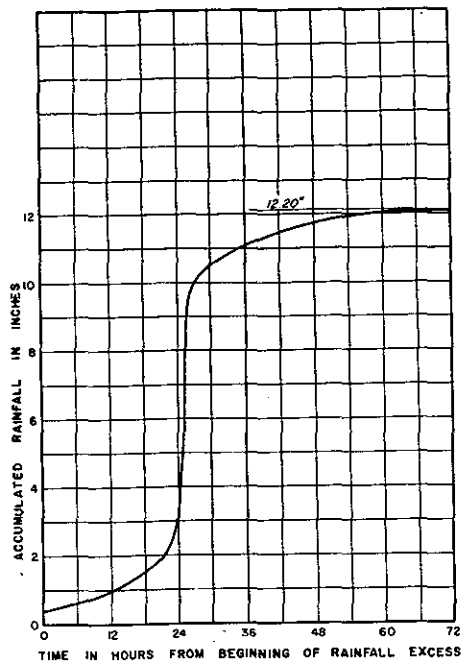
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA-DERBY, CONNECTICUT
LOCAL PROTECTION
OCTOBER & AUGUST 1955 FLOODS
NAUGATUCK RIVER, CONNECTICUT

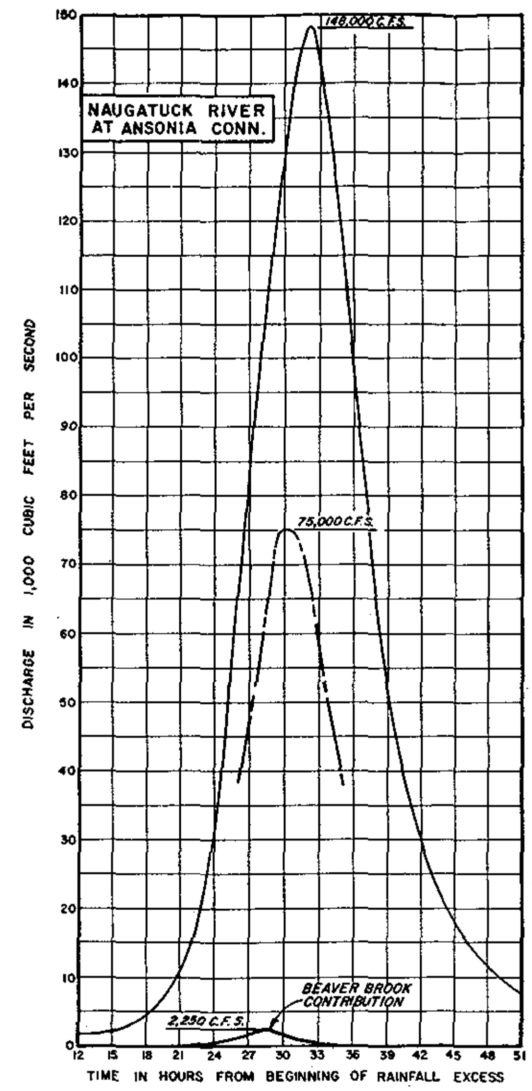
DR. BY | TR. BY | CK. BY
PROJECT ENGINEER
CHIEF, SECTION
SUBMITTED BY
CHIEF, PLAN & RPTS BRANCH
APPROVED
CHIEF, ENGINEERING DIV.
DATE APRIL 1965
SCALE
SPEC. NO. CIV ENG-18-016-
DRAWING NUMBER
SHEET

PLATE NO. I-4

CORPS OF ENGINEERS U. S. ARMY

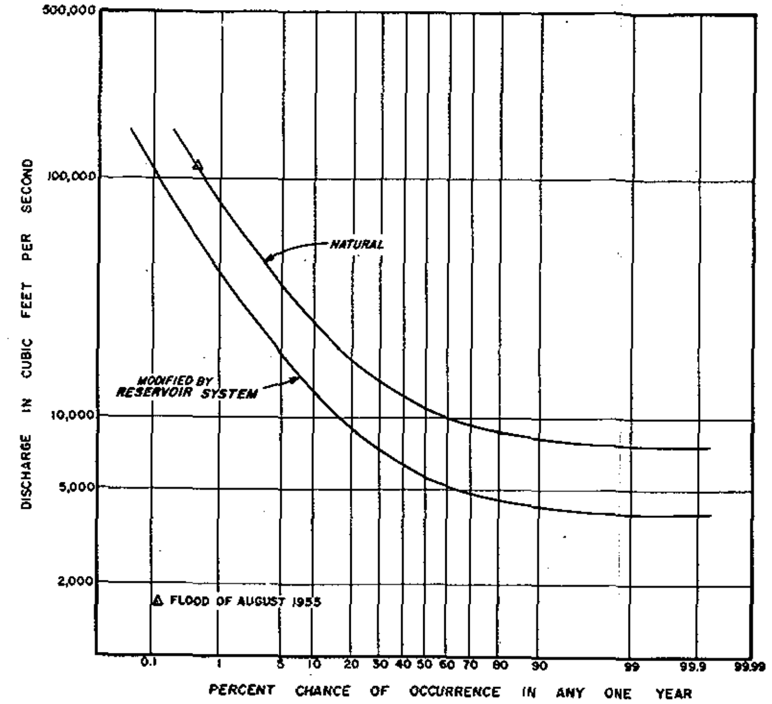


TYPICAL MASS RAINFALL CURVE
NAUGATUCK RIVER BASIN

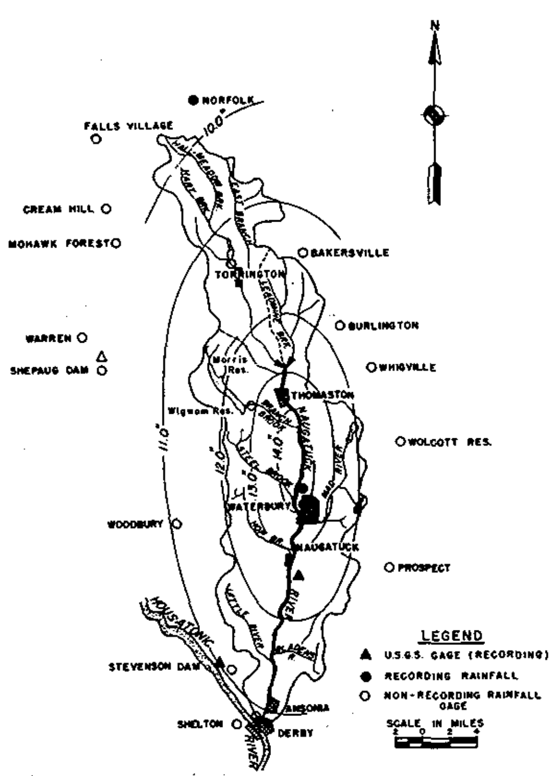


STANDARD PROJECT FLOOD
NAUGATUCK RIVER BASIN

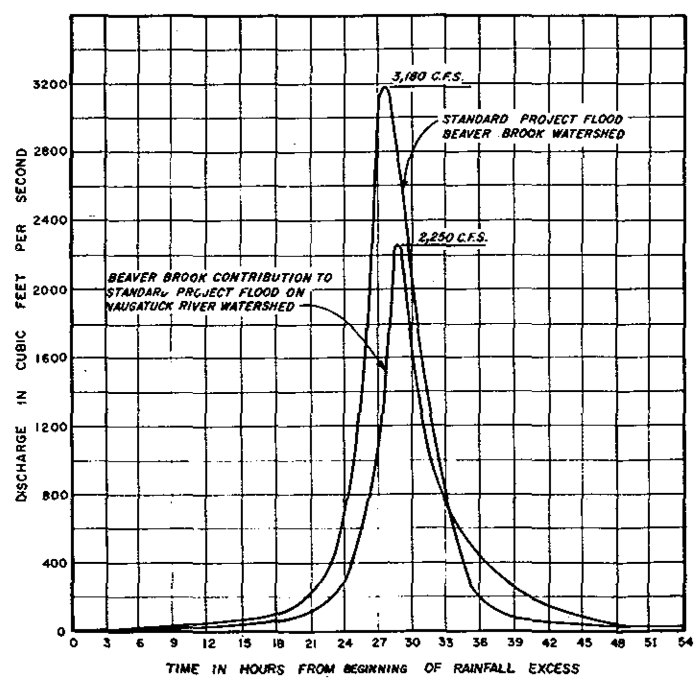
——— NATURAL
– – – MODIFIED BY COMPREHENSIVE SYSTEM OF RESERVOIRS.
THOMASTON RESERVOIR (PLUS HALL MEADOW BROOK AND EAST BRANCH RESERVOIRS)
NORTHFIELD BROOK RESERVOIR
BLACK ROCK RESERVOIR
HOP BROOK RESERVOIR
HANCOCK BROOK RESERVOIR



DISCHARGE FREQUENCY CURVES
NAUGATUCK RIVER AT MAPLE STREET BRIDGE



ISOHYETAL MAP
STANDARD PROJECT STORM
NAUGATUCK RIVER BASIN
96 HOUR DURATION



DISCHARGE HYDROGRAPHS
BEAVER BROOK NEAR CENTRAL STREET

REVISION	DATE	DESCRIPTION	BY

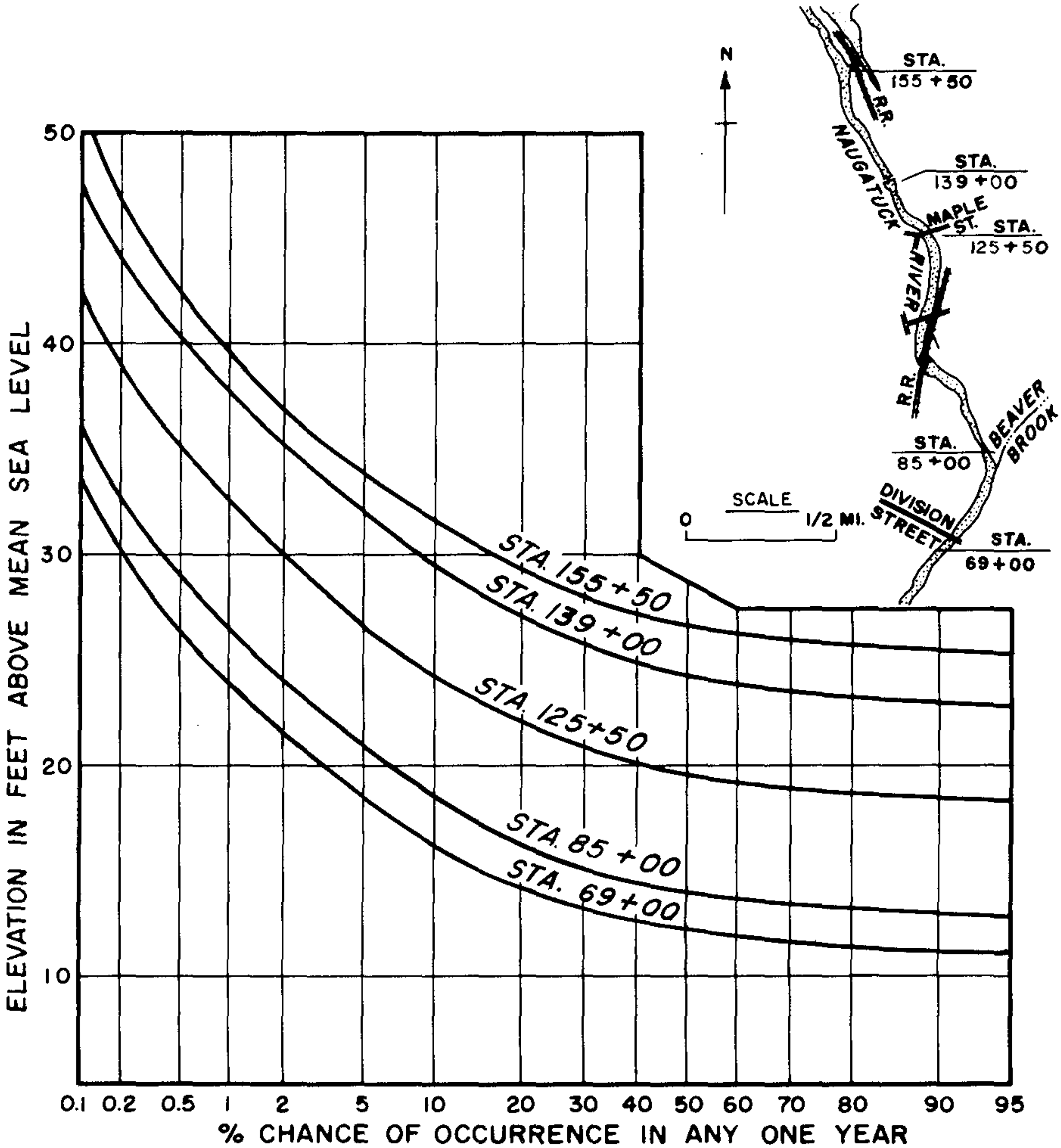
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY | TR. BY | CK. BY

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY CONNECTICUT
LOCAL PROTECTION
STANDARD PROJECT FLOODS
NAUGATUCK RIVER, CONNECTICUT

PROJECT ENGINEER
CHIEF, SECTION
SUBMITTED BY
CHIEF, PLANN. & R.P.T.S. BRANCH
APPROVED
DATE APRIL 1965
CHIEF, ENGINEERING DIV.
SCALE
SPEC. NO. CIV. ENG.-19-016-
DRAWING NUMBER
SHEET

PLATE NO. I-5



NOTES

1. Curves show effect of comprehensive reservoir system.

2. Stage frequency curves for Sta. 69+00 and Sta. 85+00 are based on stage-discharge relationships for average tidal and flood conditions on the Housatonic River.

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT

LOCAL PROTECTION

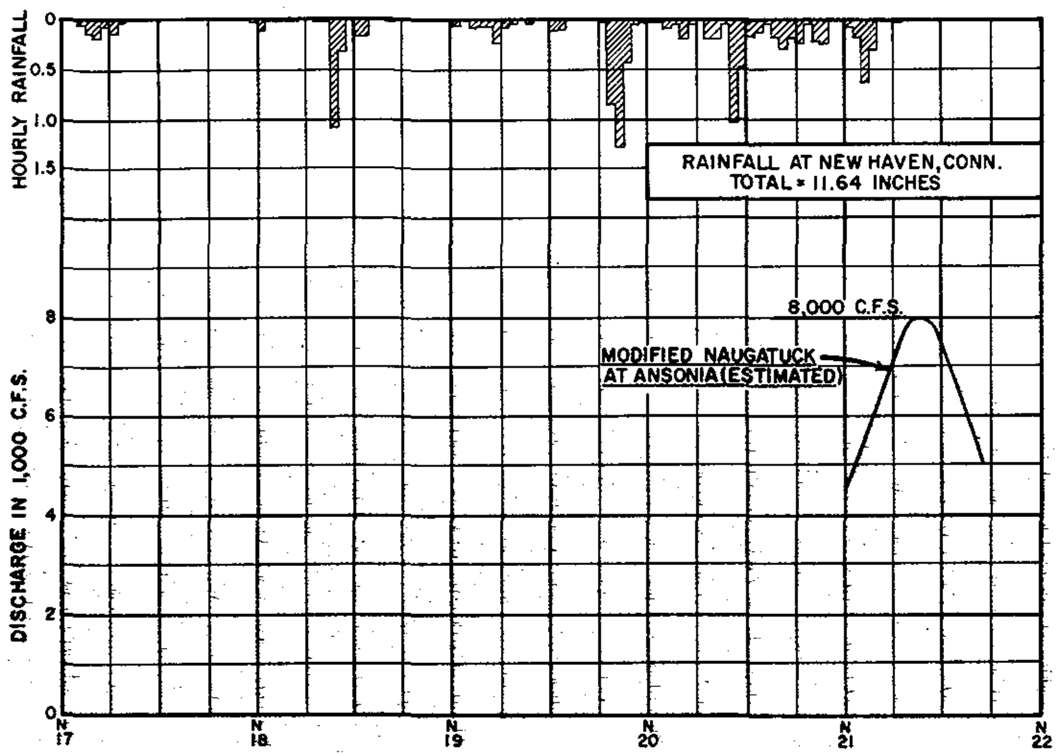
STAGE - FREQUENCY CURVES AT SELECTED LOCATIONS

NAUGATUCK RIVER, CONNECTICUT

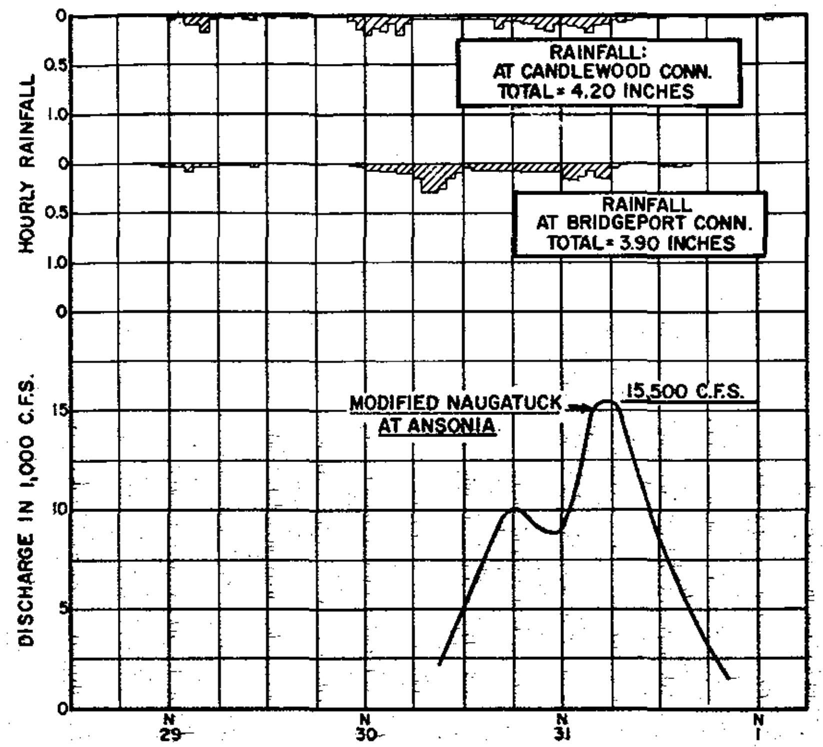
PLATE NO. 1-6

CORPS OF ENGINEERS

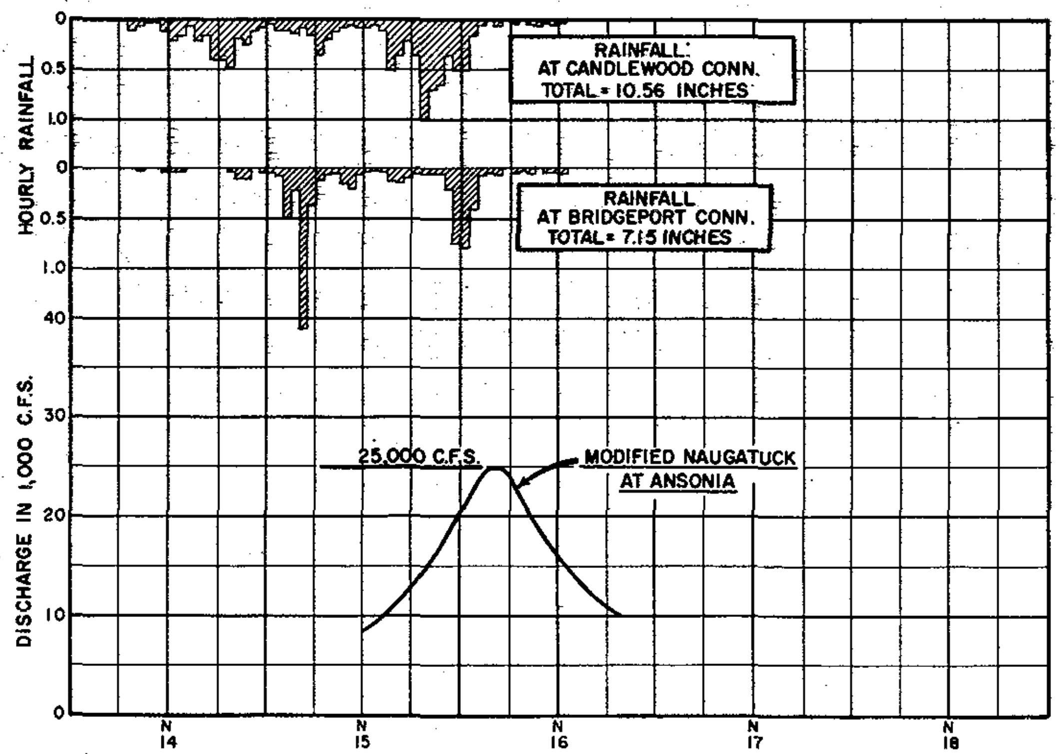
U. S. ARMY



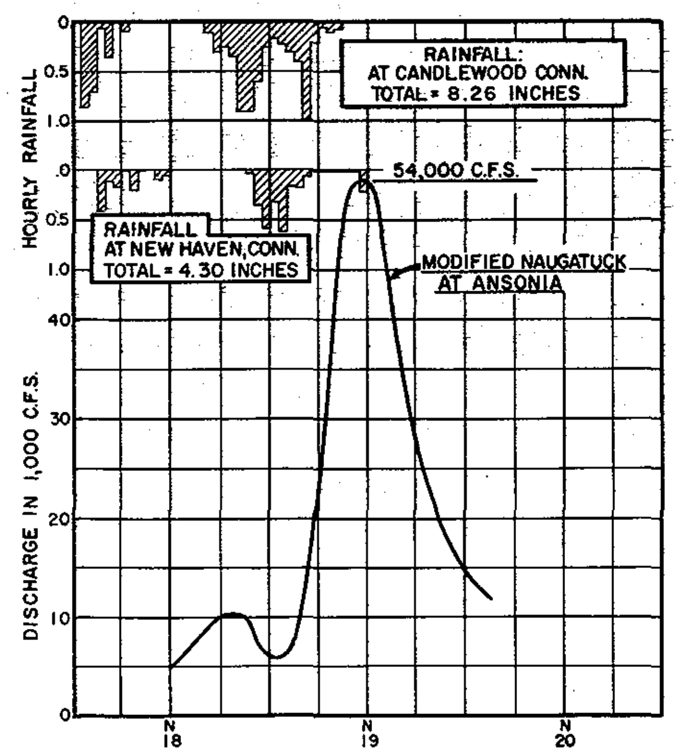
SEPTEMBER 1938



DECEMBER 1948



OCTOBER 1955



AUGUST 1955

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY | TR. BY | CK. BY

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT
COINCIDENCE OF PAST FLOOD STAGES
WITH LOCAL RAINFALL

ENGINEER
SUBMITTED BY:
CHIEF, HYD. & HYD.
APPROVED:
CHIEF, PLANN & RPTS. BR.

NAUGATUCK RIVER, CONNECTICUT
APPROVED:
CHIEF, ENGINEERING DIVISION
DATE: APRIL 1965
SCALE: AS SHOWN
DRAWING NUMBER
SHEET

PLATE NO. I-7

CORPS OF ENGINEERS U.S. ARMY

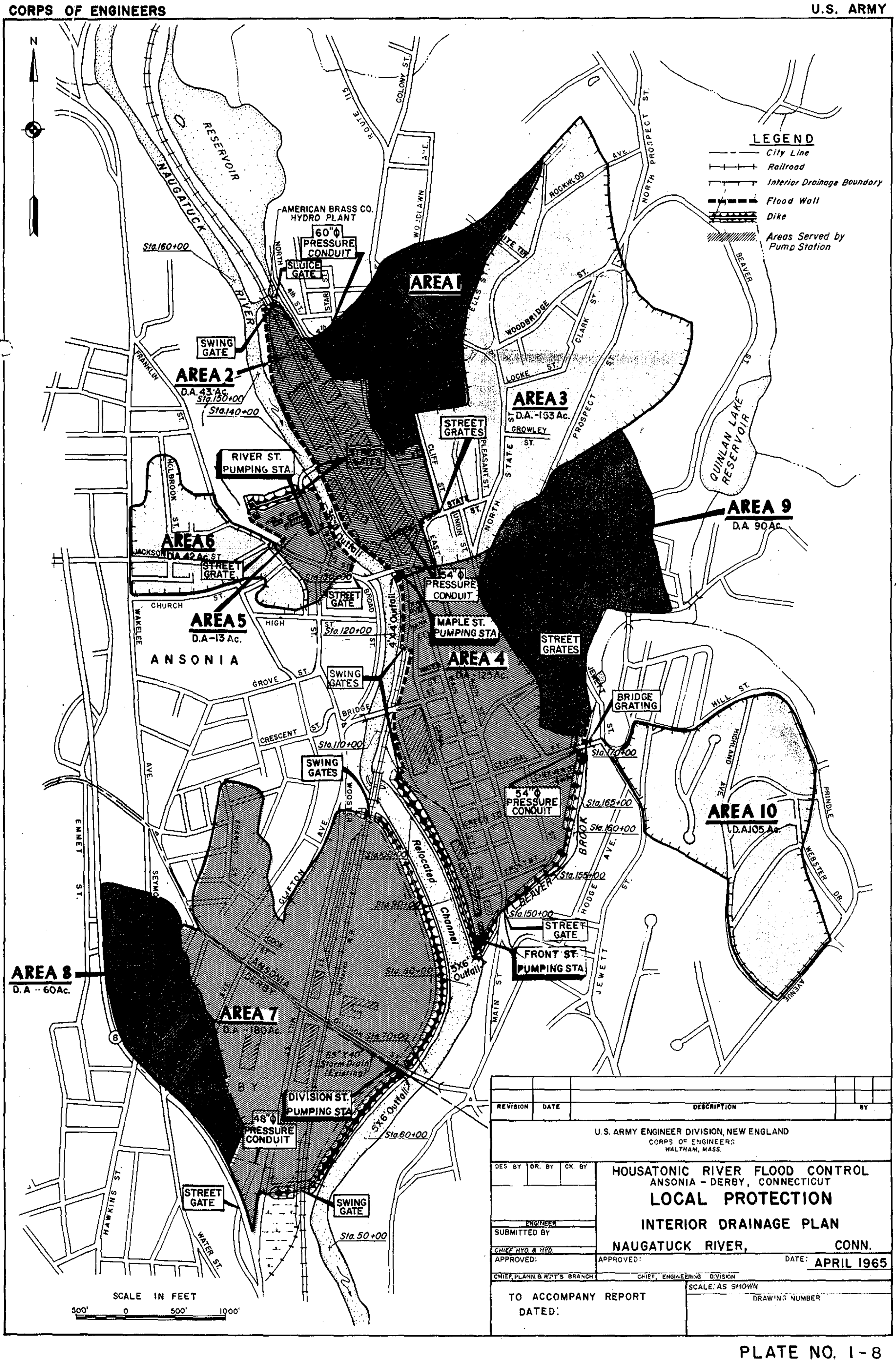
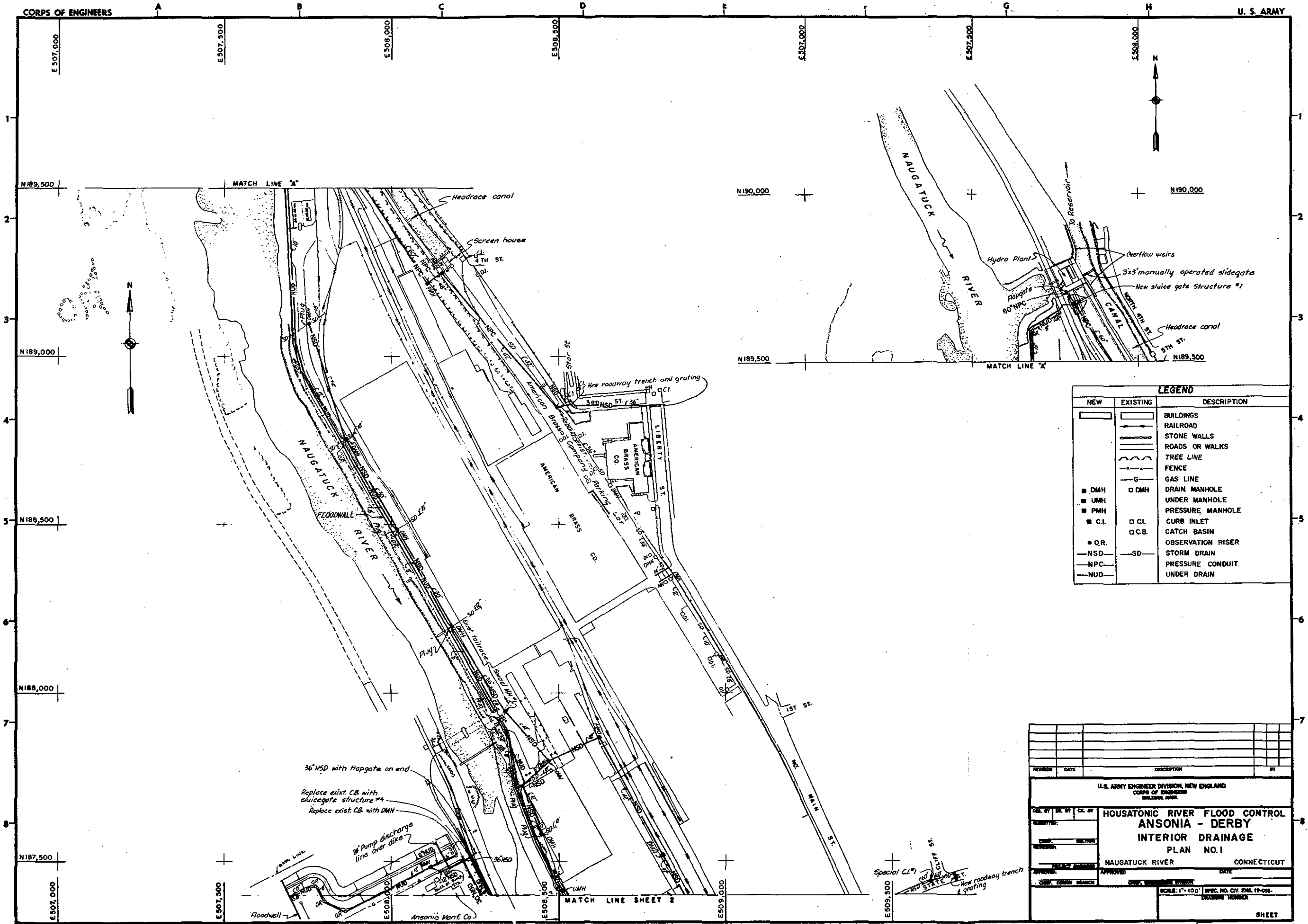


PLATE NO. I-8



CORPS OF ENGINEERS
U. S. ARMY
MATCH LINE "A"
Headrace canal
Screen house
4TH ST.
NAUGATUCK RIVER
FLOODWALL
AMERICAN BRASS CO.
American Brass Company Parking Lot
LIBERTY ST.
New roadway trench and grating
STAR ST.
Exist. tailraces
36" NSD with flapgate on end
Replace exist. CB. with sluicegate structure #4
Replace exist. CB. with DMH
24" Pump discharge line over dike
Floodwall
Ansonia Manf. Co.
MATCH LINE SHEET 2
1ST ST.
NO. MAIN ST.
Special C.I. #1
STATE ST.
New roadway trench & grating
To Reservoir
Hydro Plant
Overflow weirs
3'x3' manually operated slidegate
New sluice gate Structure #1
Flapgate
60" NPC
CANAL
NORTH 4TH ST.
5TH ST.
LEGEND
NEW
EXISTING
DESCRIPTION
BUILDINGS
RAILROAD
STONE WALLS
ROADS OR WALKS
TREE LINE
FENCE
GAS LINE
DRAIN MANHOLE
UNDER MANHOLE
PRESSURE MANHOLE
CURB INLET
CATCH BASIN
OBSERVATION RISER
STORM DRAIN
PRESSURE CONDUIT
UNDER DRAIN
REVISION
DATE
DESCRIPTION
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY
INTERIOR DRAINAGE
PLAN NO. 1
NAUGATUCK RIVER
CONNECTICUT
APPROVED
DATE
SCALE: 1" = 100'
SHEET

PLATE NO. 1-9

CORPS OF ENGINEERS U. S. ARMY

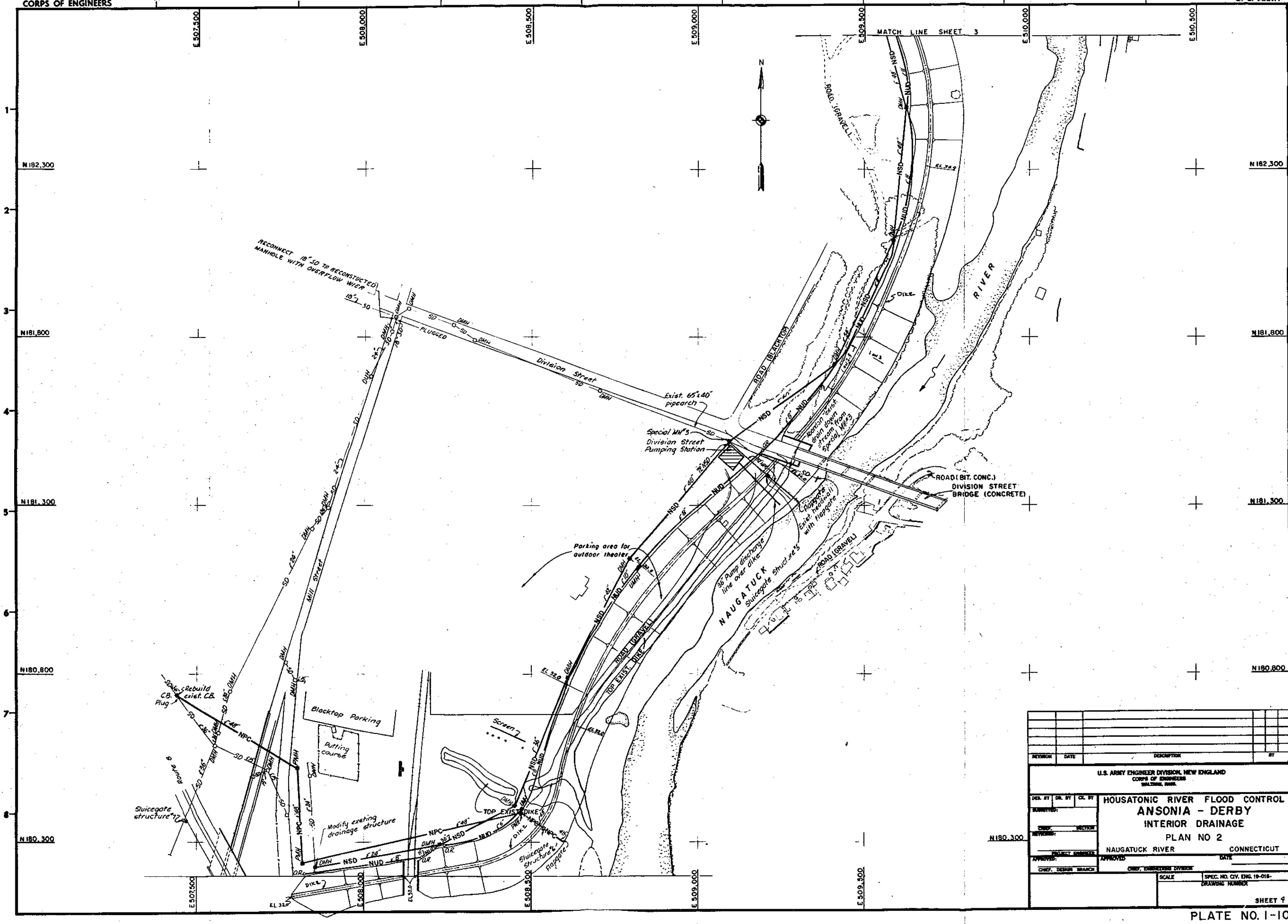


PLATE NO. I-10

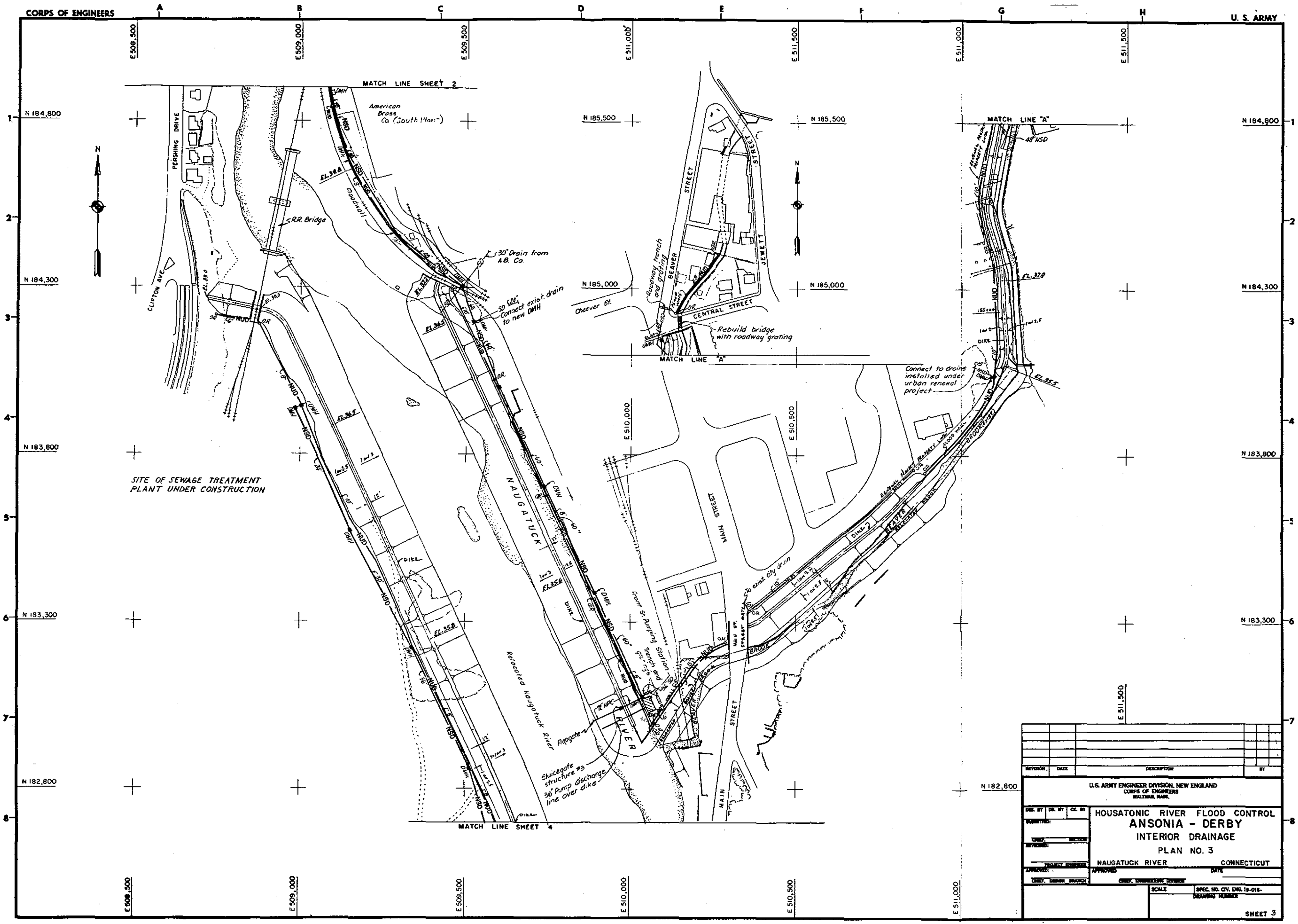
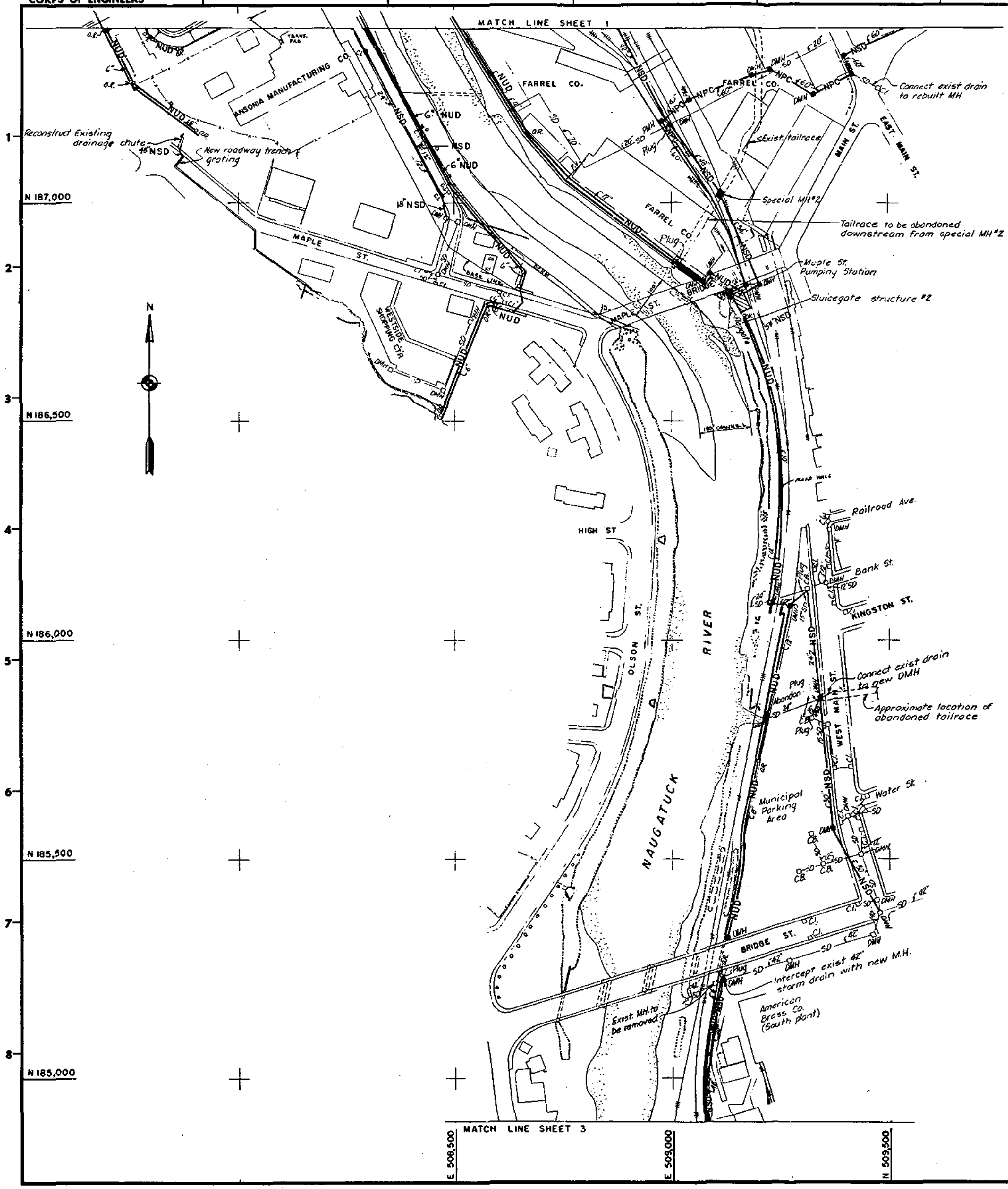

CORPS OF ENGINEERS
U. S. ARMY
MATCH LINE SHEET 2
MATCH LINE SHEET 4
MATCH LINE "A"
SITE OF SEWAGE TREATMENT
PLANT UNDER CONSTRUCTION
NAUGATUCK
RIVER
Relocated Naugatuck River
R.R. Bridge
American Brass Co. (South Plant)
30" Drain from A.B. Co.
Connect exist drain to new DMH
Rebuild bridge with roadway grating
Connect to drains installed under urban renewal project
Front St. Pumping Station
Flapgate
Sluicegate structure #3
36" Pump discharge line over dike
PERSHING DRIVE
CLIFTON AVE.
BEAVER STREET
JEWETT STREET
CENTRAL STREET
Cheever St.
MAIN STREET
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY
INTERIOR DRAINAGE
PLAN NO. 3
NAUGATUCK RIVER
CONNECTICUT
SHEET 3

PLATE NO. I-11

CORPS OF ENGINEERS | U. S. ARMY



REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY
INTERIOR DRAINAGE
PLAN NO. 4

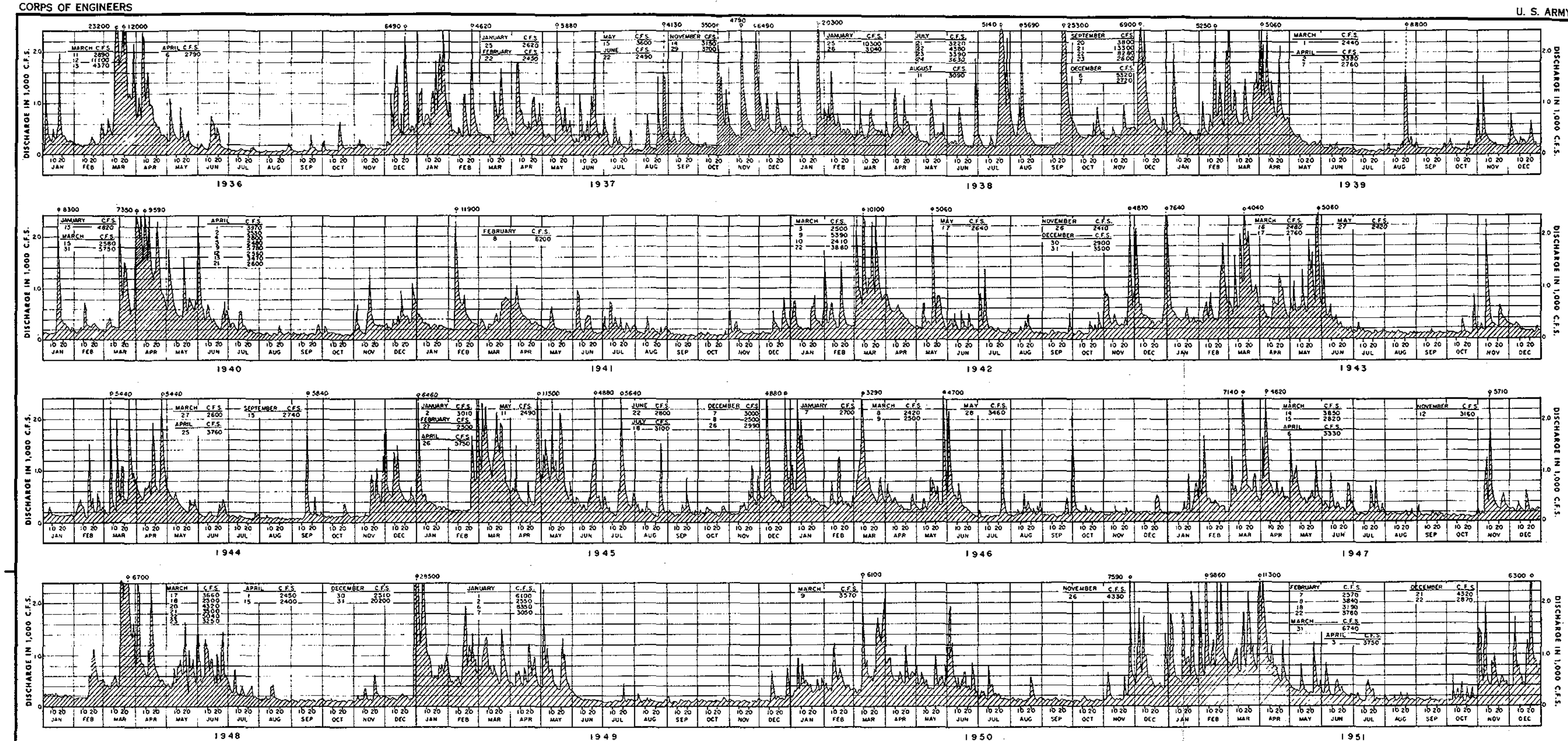
NAUGATUCK RIVER — CONNECTICUT

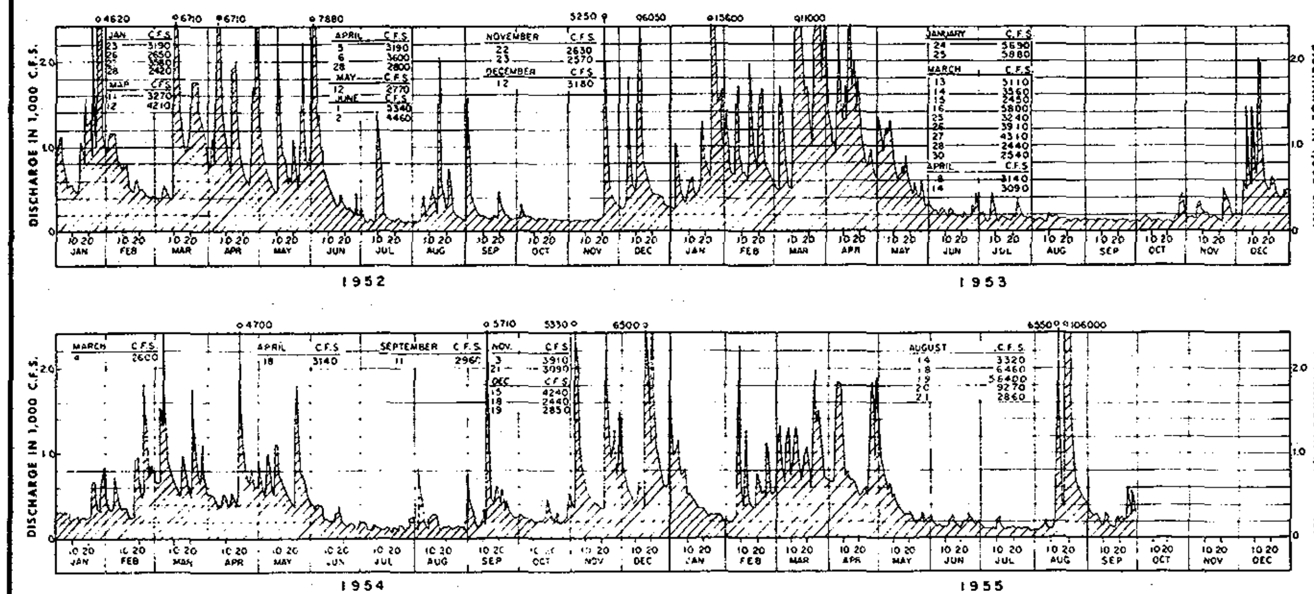
SCALE: 1" = 100'

PLATE NO. I-12

CORPS OF ENGINEERS

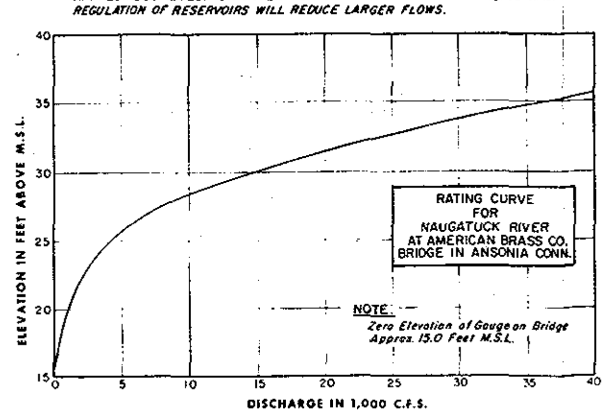
U. S. ARMY





NOTES:

1. THESE HYDROGRAPHS ARE THE DAILY AVERAGE STREAMFLOW OF NAUGATUCK RIVER AT NAUGATUCK, CONN. FROM THE TRIBUTARY DRAINAGE AREA OF 246 SQUARE MILES. THE TRIBUTARY DRAINAGE AREA OF THE NAUGATUCK AT ANSONIA IS 306 SQUARE MILES. SOME INSTANTANEOUS PEAK DISCHARGES ARE SHOWN BY ⊙.
2. REGULATION OF THE SYSTEM OF UPSTREAM RESERVOIRS WILL HAVE NO APPRECIABLE EFFECT ON RIVER FLOWS BELOW APPROXIMATELY 5,000 C.F.S. REGULATION OF RESERVOIRS WILL REDUCE LARGER FLOWS.
3. THE DATA CONTAINED HEREON ARE NOT INTENDED AS REPRESENTATIONS OR WARRANTIES BUT ARE FURNISHED AS INFORMATION ONLY. IT IS EXPRESSLY UNDERSTOOD THAT THE GOVERNMENT WILL NOT BE RESPONSIBLE FOR ANY DEDUCTION, INTERPRETATION OR CONCLUSION THEREFROM MADE BY ANY BIDDER OR CONTRACTOR.



REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY | TR. BY G.H.D | CK. BY

PROJECT ENGINEER

CHIEF, SECTION

SUBMITTED BY

CHIEF, PLANN. & RPTS. BRANCH

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT

HYDROGRAPHS

NAUGATUCK RIVER, CONNECTICUT

APPROVED

CHIEF ENGINEERING DIV.

DATE APRIL 1965

SCALE | SPEC. NO CIV. ENG.-19-016-

DRAWING NUMBER

SHEET

PLATE NO. 1-13